SCHOOL OF CIVIL ENGINEERING

INDIANA DEPARTMENT OF HIGHWAYS

JOINT HIGHWAY RESEARCH PROJECT

FHWA/IN/JHRP-85/16

Interim Report

BEHAVIOR OF MUCKS AND AMORPHOUS PEATS AS EMBANKMENTS FOUNDATIONS

Paul G. Joseph

PURDUE UNIVERSITY

JOINT HIGHWAY RESEARCH PROJECT

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TO: H.L. Michael, Director
Joint Highway Research Project

August 28, 1985

FROM: C.W. Lovell, Research Engineer
Joint Highway Research Project

File: 6-6-16
Project: C-36-5P

The attached report is an interim one describing study of Indiana mucks and amorphous peats as potential embankment foundations. It is authored by Paul G. Joseph, formerly a research assistant on our staff.

It reviews and recommends procedures for classifying, sampling, and testing such materials. A significant part of the report deals with the finite strains, nonlinearity of the material properties, and the intrinsic time effects associated with such materials. All of these are tied together into theories, and two of these theories are recommended for use.

One predictive theory is too complex for routine use, but the other may have wide applicability, when combined with simple construction observations.

This report does not lead to a simple design/construction routine, and accordingly, the study has been recommended for extension...to permit the development of such a routine.

The report is submitted for review, comment and acceptance in partial fulfillment of the referenced study.

Respectfully submitted,

C. W. Lovell/HLM

C.W. Lovell
Research Engineer

CWL/kr

Interim Report

BEHAVIOR OF MUCKS AND AMORPHOUS PEATS AS EMBANKMENT FOUNDATIONS

by

Paul G. Joseph

Graduate Instructor in Research

JOINT HIGHWAY RESEARCH PROJECT
File: 6-6-16
Project: C-36-5P

Prepared as Part of an Investigation

Conducted by

Joint Highway Research Project
Engineering Experiment Station

Purdue University

in cooperation with the

Indiana Department of Highways

and the

U.S. Department of Transportation
Federal Highway Administration

Purdue University
West Lafayette, Indiana

August 28, 1985

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/IN/JHRP-85/16		

4. Title and Subtitle	5. Report Date / 6. Performing Organization Code
BEHAVIOR OF MUCKS AND AMORPHOUS PEATS AS EMBANKMENT FOUNDATIONS	5. Report Date: August 28, 1985 6. Performing Organization Code:
7. Author(s) Paul G. Joseph	8. Performing Organization Report No. JHRP-85-16
9. Performing Organization Name and Address Joint Highway Research Project Civil Engineering Building Purdue University West Lafayette, IN 47907	10. Work Unit No. 11. Contract or Grant No. HPR-1(23) Part II
12. Sponsoring Agency Name and Address Indiana Department of Highways State Office Building 100 North Senate Avenue Indianapolis, IN 46204	13. Type of Report and Period Covered Interim Report 14. Sponsoring Agency Code

15. Supplementary Notes

Prepared in cooperation with the U.S. Department of Transportation, Federal Highway Administration on HPR Part II Study "Behavior of Mucks and Amorphous Peats as Embankment Foundations".

16. Abstract

The materials mucks and amorphous peat are among the poorest to use as an embankment foundation. Yet, if they are loaded in a careful manner, with accompanying field observation of movements and pressures it is possible to build embankments over them. This may be an economic alternative to the conventional excavation and replacement techniques.

This report reviews the behavior of the above materials under compressive loading, and observes that their response is very similar to very soft clays. Since the latter materials have been intensively studied, there is a considerable volume of technical information already available for the prediction of behavior of mucks and amorphous peats. The author suggests procedures for their: classification, sampling and strength and compression testing. He further proposes prediction theories (for the performance of compacted highway embankments over them) at two levels of complexity. It is likely that the more simple theory is adequate, and it is proposed that a simple design/construction routine be developed in conformance with it. This simple procedure is the subject of a proposed extension to this research.

17. Key Words	18. Distribution Statement
Peat, Muck, Organics, Settlement, Classification, Sampling, Rate of Loading	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	309	

Form DOT F 1700.7 (8-69)

ACKNOWLEDGEMENTS

The author would like to thank his major advisor, Professor C. W. Lovell, for his guidance in preparing this thesis. He would also like to thank Mr. An-Bin Huang, for his advice and recommendations, and for the use of the K_o triaxial cell that he fabricated. The author would like to thank Ms. J. Lovell for the use of her laboratory. The author wishes to thank Ms. Catherine Ralston for her prompt, accurate typing and Ms. Marin Gomez for her drafting. The author expresses his gratitude to his friends, especially Mr. Yahia Mohammed Zein and Mr. Anil Kollipara, for their help and support. Mrs. Barbara Beaver has been of immense help in the preparation of the thesis, for which the author is grateful.

Financial support for this research was provided by the Indiana Department of Highways and The Federal Highways Administration. The research was administered through The Joint Highway Research Project, Indianapolis, Indiana. Charles W. Lovell, Jean-Lou Chameau and Donn E. Hancher were on the author's committee.

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LIST OF SYMBOLS AND ABBREVIATIONS

a	- initial (Lagrangian) coordinates, also rheological parameter in Gibson-Lo's model
a_{v_o}	- initial coefficient of compressibility
ASTM	- American Society of Testing Materials
B	- Skempton's parameter, also base width of embankment
b	- rheological parameter in Gibson-Lo's model
C_c	- compression index
C'_c	- modified compression index
c_v	- coefficient of consolidation
C_k	- slope of the e-log k line
c_v	- coefficient of vertical consolidation
$C\overline{IU}$	- consolidated isotropically and sheared undrained with pore pressure measurement
$CK_o\overline{U}$	- consolidated under K_o conditions and sheared undrained with pore pressure measurement
CKU-AC	- consolidated under K_o conditions, and sheared undrained in axial compression
C_α	- coefficient of secondary compression $\Delta e/\Delta \log t$
^oC	- degree centigrade
D_f	- depth to failure surface
D_i	- distance corresponding to column i
D	- dilatency energy under unit normal force

e	- void ratio
e_{ss}	- void ratio at start of shear
$\dot{e}$	- change in void ratio with time
e_o	- initial void ratio
e_f	- final void ratio
e_{10}	- initial void ratio corresponding to consolidation
e_{1f}	- final void ratio corresponding to consolidation
e_{20}	- initial void ratio corresponding to creep
e_{2f}	- final void ratio corresponding to creep
e_p	- void ratio at the end of primary consolidation
Eu	- undrained Young's Modulus
f	- force
H_o	- initial thickness of layer
H_{field}	- thickness of a soil stratum
H_j	- degree of humification
H_{lab}	- height of laboratory sample
h	- Planck's constant
i	- hydraulic gradient
k	- coefficient of permeability, also Boltzman's constant
k_o	- initial coefficient of permeability
k_f	- final coefficient of permeability
K_o	- coefficient of lateral earth pressure at rest

$K_{o_{OC}}$ - K_o for overconsolidated soil

$K_{o_{NC}}$ - K_o for normally consolidated soil

Kg - kilogram

kNt - kilo Newton

kPa - kilo Pascal

L - thickness of layer

lb - pound

m - meter

mm - millimeter

n - porosity

$\dot{n}$ - change in porosity with respect to time

N_i - weight corresponding to column i

OCR - overconsolidation ratio

p - confining pressure $(\sigma_1+\sigma_2)/2$

p′ - effective confining pressure

P - normal force

pH - negative log to the base 10 of the Hydrogen ion concentration

psi - pound per square inch

P_A - normal stress corresponding to point A in Fig. 6.2

P_B - normal stress corresponding to point B in Fig. 6.2

P_C - normal stress corresponding to point C in Fig. 6.2

Pt - peat

q	- shear stress $(\sigma_1 - \sigma_3)/2$
Q	-load
$\dot{q}$	- loading rate
R_j	- value of the rule 'j'
S_{SEC}	- settlement due to secondary compression
t	- time
t_p	- time at end of primary compression
T	- dimensionless time, also absolute temperature
u	- pore pressure
u_e	- excess pore pressure
v_f	- apparent fluid velocity
$\bar{v}_f$	- real fluid velocity
v_s	- apparent solid velocity
$\bar{v}_s$	- real solid velocity
VPHS	- von Post Humification Scale
w_{ss}	- water content at start of shear
x	- current coordinate
y_0	- initial yield locus
y_1	- aged yield locus
z	- reduced coordinate system
$\alpha(e)$	- rheological parameter
$\beta(e)$	- rheological parameter
β_o	- rheological parameter
β_s	- rheological parameter
δ	- lateral displacement

Δe	- change in void ratio
ΔE	- activation energy
ΔE_o	- stress independent physico-chemical activation energy
ΔQ	- change in load
Δs	- change in vertical settlement
Δn	- a small length
Δy_m	- maximum lateral displacement
$\Delta \delta$	- change in lateral displacement
$\Delta \sigma$	- change in normal stress
$\Delta \sigma_{oct}$	- change in octahedral normal stress
ε	- strain
ε_{fail}	- strain at which failure occurs
$\dot{\varepsilon}$	- strain rate
γ_w	- unit weight of water
γ_s	- unit weight of solids
λ	- rheological parameter
μ	- dimensionless pore pressure
$\nu_{\rightarrow}$	- frequency of displacements to the right
$\nu_{\leftarrow}$	- frequency of displacements to the left
ϕ	- stress dependent bond energy under a unit normal force
ϕ'	- effective friction angle
ρ_j	- settlement at time t_j
σ	- normal stress
σ'	- effective normal stress
$\bar{\sigma}$	- effective normal stress

σ_c - critical normal pressure

$\overline{\sigma}_f$ - final effective normal stress

σ_i - initial normal stress

$\overline{\sigma}_p$ - preconsolidation pressure

$\overline{\sigma}_{vf}$ - final effective vertical pressure

τ - shear stress

HIGHLIGHT SUMMARY

Peats and mucks have always been considered as problem materials from a geotechnical viewpoint because of their low shear strength, their high compressibility, creep, and variability. This report is to aid those engineers who have to deal with peats and mucks. After reviewing existing classification systems, the report suggests a modified system. A new technique, of classification, is recommended. Next, various methods of sampling peat and mucks, and techniques used in preparing samples for accurate laboratory testing, are described. A simple method (developed during the course of the testing program) for obtaining relatively undisturbed samples for testing is presented.

A significant part of the report deals with the finite strains, nonlinearity of the material properties, and the intrinsic time effects associated with such materials. Finite strain, nonlinear one-dimensional consolidation theory is examined and a review of secondary time effects made. Both of these are tied together into theories accounting for all of the above mentioned phenomena. The applicability of these theories,

both in the laboratory and in the field, is studied. Next, the laboratory testing of peats and mucks for geotechnical purposes is presented, as is the result of various index tests, consolidation tests, creep tests, permeability tests and K_o triaxial shear strength tests conducted during the testing program.

These tests were conducted on three materials, one a peat and the other two, mucks, sampled from various sites in Indiana. The observed behavior of embankments on such materials both during and after construction is described. Next, a complete design methodology for construction on such materials is presented.

Basically, the methodology involves the tying together of predictive and observational models. Two predictive models for settlement are available. One model is complex, while the other is a simplified version of the more complex model. The two observational methods suggested are the model suggested by Gruen and Lovell (1983), and the model suggested by Sekiguchi and Shibata (1979). Finally, guidelines regarding the rate of loading are presented, along with a procedure to be used in determining the geometry of the embankment.

CHAPTER I - INTRODUCTION

INTRODUCTION

The entire range of soils can be subdivided on the basis of organic contents. Such a subdivision is shown in Table 1.1. Many of the organic silts and clays, with organic contents ranging anywhere below twenty to twenty-five percent can be used as embankment foundations, with the application of conventional soil mechanics theory for design. An example of such an analysis can be found in the report by Goodman, Chameau and Lovell (1983). At the other end of the scale are materials with organic contents greater than seventy-five percent. These are the peats. As described by Gruen and Lovell (1983), there are several different types of peat. All of these different types can be broadly classified as being fibrous peats or amorphous peats.

In their study, Gruen and Lovell (1983) showed that fibrous peats could be surcharged and compressed into becoming a relatively strong and competent foundation. They also suggested a procedure through which settlements could be controlled using preloading. By adapting the Gibson-Lo (1960) model of soil deformation, they described a procedure to estimate settlements under the design load, the amount of surcharge that would be

Table 1.1 Grouping of Organic Materials.
Tentative ASTM Standard.

Ash Content %	Material Description
0 to < 25	PEAT
25 to < 50	peaty muck
25 to 75	MUCK
50 to 75	silty or clayey muck
> 75	ORGANIC SILT OR CLAY
75 to < 90	highly organic silt or clay
90 to 100	slightly organic silt or clay

required, and also the duration for which the surcharge should remain, in order to control settlements.

The materials that remain to be considered are the amorphous peats and also the mucks (Table 1.1). Relatively little is known about these materials. One reason is because they have often been misclassified as peats. Another reason is that these materials too are like peat. They too exhibit high compressibilities, a very low-shear strength, and a high variability. As a consequence, the highway engineer would avoid these types of deposits whenever possible. If the deposits are small, excavation of the material and replacement with a more desirable foundation material is sometimes feasible. However, in most cases, this is not economical. In such situations, various construction procedures such as floating the road over the deposit or supporting it on piles or preloading may be used. In general, the acceptable procedure is to preload using a surcharge. There have been cases where geotextiles have been used, ex. Fowler (1985), but there are no details available about the long term behavior, particularly with regard to creep.

As a result of this lack of construction on peats and mucks, engineers are not familiar with their properties and behavior, and tend to assume that their behavior is fundamentally different from that of other soils. The reason for this assumption is two-fold. The first reason is that differences in predicted and observed behavior were found to be high, and the second reason is that case histories of embankments on such materials describe significantly different behavior in each case, though the foundation material is uniformly labeled as "peat".

In the first case the discrepancies are due to the fact that the normally used conventional theories do not account for factors such as varying compressibility, varying permeability and creep. While these factors are present in most soils to some extent, in the case of mucks and peats they are very significant and influence behavior significantly. Hence any theory that does not account for these factors is not likely to yield good predictions. This indicates that a generalized soil model that accounts for all these factors should be capable of simulating the behavior of these soils under loading. The reason for the differences in behavior of embankments on peat is that though called "peat", the actual material may vary considerably from an amorphous peat to a clayey muck and hence, wide variations are only to be expected. Gruen and Lovell (1983) recognized this and called for a more accurate and usable classification system.

Another important factor to be considered when dealing with peats and mucks is that being highly compressible and with a low shear strength, it is very difficult to sample them without disturbance. Further, preparing and setting up samples for testing is difficult, and to be done properly requires special equipment. Most laboratories lack this equipment and hence, the results of their tests cannot be considered as being truly representative of field behavior. Compounding this factor is the high variability of these materials in both horizontal and vertical directions. Thus, the problem of how far the sample being tested is representative of the deposit also arises. Though this is true for all soils, for peats and mucks it assumes greater importance.

Some systems classify soils with organic material into two groups, namely sedimentary peats and sedentary peats. Sedimentary peats are better described as organic soils, as their mineral content may be quite high. The sedentary peats are the "true peats", and consist mostly of remains of vegetable matter together with a little mineral matter deposited by wind or water. Since there appears to be no clear cut demarcation between the two, this terminology is not used in this report. The classification used here is based on organic content, and is as shown in Table 1.1.

During the initial stages of the project it was felt that a finite element program based on an elasto-plastic model with an empirical creep function was the approach to be followed. However, as testing proceeded it soon became apparent that such an approach would not be practical. The reasons for this were, as previously mentioned, the difficulty of testing undisturbed samples and the high variability of such soils. Because of these reasons it would be extremely difficult to select the correct values of the parameters involved for input into the program.

Apart from the finite element approach was the approach based on the solution of the finite strain nonlinear one-dimensional consolidation equation. For the same reasons as mentioned above, it was felt that this approach too could not be justified, except possibly for test fills; the purpose being to further the state of the art. On the other hand, it was understood that a purely observational procedure would not be justifiable from a planning point of view. As a result of this, it was felt that an approach of the type suggested by Gruen and Lovell (1983) for fibrous peats would be appropriate for amorphous peats and mucks also.

A simplified method of calculating settlements, while accounting for nonlinearity, finite strains, and creep behavior, is suggested. This approach is based on laboratory tests and is used to get an estimate of the magnitude of settlements to expect. These same tests can also be used to obtain the parameters necessary for the approach using the Gibson-Lo model. Based on these calculations, a prediction of the settlement is made. Once the surcharge is placed, the observational method is used to constantly re-evaluate the parameters involved.

Gruen and Lovell (1983) showed that fibrous peats could be surcharged and compressed into a relatively strong and competent foundation material. It was felt that similar behavior would be exhibited by these materials.

Keeping all these factors in mind, it was felt that a critical evaluation of the geotechnical properties of peats and mucks would be necessary. This evaluation would be based both on a search of existing literature and also the results and experience obtained from testing. Three materials, with appropriate organic contents, were sampled undisturbed from the field. They are described in detail elsewhere. After running index tests, a series of consolidation tests were run to define the e-log curve. A series of creep tests were run, as well as a series of permeability tests and undrained shear strength tests with pore pressure measurement on samples consolidated under K_o conditions ($CK_o\bar{U}$).

SCOPE

The report starts with an evaluation of existing classification schemes. A unified classification for peats is suggested. Also suggested is an approach for classification, using fuzzy sets, that offers very significant and practical advantages, and has been brought to the attention of D18.18. This subcommittee of ASTM, has jurisdiction over standards for peats and highly organic soils.

Next comes site investigation, undisturbed sampling and preparation of samples. A sophisticated sampling technique, developed specifically for peats is described. As this approach is relatively complicated, a simplified approach developed during the course of the experimental program is also presented. Next follows a comprehensive theoretical investigation into the compression behavior of these materials. After developing the rigorous equations that apply, and describing the two existing theories about creep, two comprehensive theories accounting for finite strains, nonlinearity, and creep are presented. A simplified approach is also explained that accounts for the above factors. The practical application of these methods is then studied, and finally, an analysis of the scaling to be used for the field time for consolidation based on laboratory time for consolidation is presented.

The next section describes the materials tested and the tests run. The tests performed were: index tests, consolidation tests, creep tests, permeability tests, and $CK_o\bar{U}$ tests. The $CK_o\bar{U}$ tests require specialized testing equipment and were run because the experience of the previous investigators was that conventional $CI\bar{U}$ tests have in general not been too successful. Next comes a comprehensive report on the behavior of embankments founded on such materials. It was felt that this would be of aid to the engineer, providing some indication as to what behavior to expect, so that anomalous behavior could be observed and reported. The next section deals with observational methods, and presents a design methodology to be used with such materials. Finally, conclusions and recommendations for future work are [illegible] derivation of the finite strain nonlinear one-dimensional equation, and a questionnaire used to study existing classification systems for peats are presented.

CHAPTER II - CLASSIFICATION OF PEATS AND MUCKS FOR GEOTECHNICAL ENGINEERING

INTRODUCTION

Even a cursory survey of the literature indicates that there exists a lot of confusion about the classification of materials containing organics. It is not uncommon to find soils with mineral contents as high as sixty to seventy percent being classified as peat. The reason for this is that in some terminologies, Landva (1980), organic soils are referred to as sedimentary peats. These sedimentary peats include such materials as ooze, gyttja, dy, slime, mud, detritus, etc. However, these terms are relevant only locally and are not universally known. Further, there seems no clear cut demarcation between the sedimentary peats and the sedentary peats. The sedentary peats are the "true peats".

The reason for this considerable confusion is because there are many groups, each with its specific and particular reason for dealing with such materials. Some groups are interested in peats and mucks from an agricultural, forestry or horticultural point of view, while others consider such materials as a fuel source, and so on. Radforth (1952) (1969) was the first to render the system suitable for engineering applications. In order to clear up the prevailing confusion, the American Society for Testing Materials (ASTM), started working on a classification system that would apply to all relevant areas. They produced a tentative standard for grouping the materials, shown in Table 1.1, and based on organic contents. This terminology is strictly followed in this report, as it is considered to be an easy-to-use system that is universally applicable.

Within the framework of the divisions shown in Table 1.1, further classification was required. The reason for this was that each division included several types of soils, each with its own specific characteristics. Thus, the division ´peats´ included various types such as amorphous peats or fibrous peats. Burwash and Wiesner (1984) and Gruen and Lovell (1983) recognized the need for a classification system for peats. A considerable part of the confusion regarding the behavior of such materials under load can be traced to a faulty classification of the materials involved. It is only to be expected that a clayey soil with about 30% organics will behave in a manner significantly different from a material that is 90% fibrous organics. Hence, it is obvious that when both these materials are called peat, considerable confusion will arise.

It is clear from the above discussion that a comprehensive classification system is needed for materials which contain organics. Burwash and Wiesner (1984) pointed out that in order to recognize what type of material was encountered on a given project, geotechnical engineers would require straight forward guidelines. Further, in order to facilitate comparison and communication of test results and observations to others engaged in similar work, a logical basis for description of the peats encountered was needed. Finally, geotechnical engineers would need to know how accurately a peat should be classified to meet project schedules. A good classification system would satisfy all these requirements. While these comments were made about peats, the statements made hold also for mucks.

NEED FOR CLASSIFICATION

Until recently, geotechnical engineers used various methods for classifying peats. Some of these classification methods had in fact been borrowed from botanists and others. Until recently, as only a few geotechnical engineers had to deal with peat, this was not a serious problem. However, the number of construction projects involving such materials has increased.

The simplest system for classifying organic materials in general is based on color. Such a classification system was not found useful. Another method was to classify the soil according to its origin. Examples of such classifications are - Irish peats, Indiana mucks, etc. This method of classification too is not useful, as all the Indiana mucks for example do not have the same properties. One more method of classification is that used by botanists which classifies peat into groups based on their major botanical component. This system has not become popular with engineers since engineers are not trained to identify plants. This is compounded by the fact that decomposition renders identification more difficult and increases the chances of error.

Radforth (1952) proposed a complex classification system. The system consists of two parts, and depending on the complexity of the problem, engineers can choose to use either the whole system or only one part. The first part is the identification and classification of the surface vegetation on the basis of texture and height. The second part of the system involves classification of the actual soil. Within this part of the system there are seventeen subdivisions. The two parts of the system are connected to a certain extent. More details can be found in the report by Gruen and Lovell (1983). The first part of the system is useful because from the classification of the terrain it is possible to determine what sort of access is possible. Thus, route selection can be assessed fairly easily. The second part of the system, however, is not widely used because it is too complex. Identification and classification of a material using this system would require that the person concerned be an expert in using the system. Also the question arises of whether there is any need for so many different subdivisions.

Another classification scheme that has found use is the Humification Scale suggested by von Post (1922). The classification is based on a field test which is relatively simple. Some success has been achieved in correlating engineering properties with the degree of humification.

All of the classification systems described above have had their uses. For current geotechnical purposes, however, they are not adequate. In 1983, a new version of ASTM D2487 was approved. As this consisted of a thorough revision and updating of the inorganic sections of the Unified Soil Classification System, it was felt that a similar revision for peats and mucks would be appropriate.

At present the test used to distinguish between organic and inorganic soils is the liquid limit test. The criterion is that if the liquid limit of the oven dried soil is less than three fourths of the liquid limit of the undried soil, the soil is organic. The basis of this test is that organic particles will not absorb back as much water as they lost upon heating. However, Arman (1970) ran tests on artificial mixtures of soil and organics, and his data showed no consistent or definite trend for the affect of the percent organics on the liquid limit, the plastic limit or the plasticity index. Also, he found that the Atterberg limits were meaningless for soils with more than 30 percent organic content. Another important conclusion by Arman was that for organic contents of less than twenty percent, behavior is influenced primarily by the mineral particles. As the organic content goes above 20 percent, the influence of the organic matter on the engineering properties increases. The effect of the organics also depends on the form of the organic matter, which may range from a gel-like substance to fibers, stems and leaves.

As previously mentioned, the tentative ASTM standard for classifying organic materials is shown in Table 1.1. The ASTM Test Method for Classification of Soils for Engineering Purposes (D2487) was significantly revised in 1983. Among the changes and revisions adopted was the redefinition of organic silts and clays.

CLASSIFICATION OF MUCKS

The organic soils were divided into two groups, namely organic silt (OL) and organic clays (OH). In ASTM D2487-69, these soils could occur only below the "A" line. If the liquid limit was less than 50, it was classified as an OL, otherwise it was an OH. However, it was found that some organic soils plotted over the "A" line. According to Casagrande (no date), originally, the "A" line was defined as an empirical boundary between typical inorganic clays and plastic organic soils. Casagrande stated that he was not aware of the existence of organic clays which fell above the "A" line. Except for a substantial loss of plasticity when drying, these soils had substantially the characteristics of inorganic clays. It was suggested that the "A" line be moved so that all these soils fell below it. This, however, would bring all the inorganic soils below the "A" line. Casagrande felt that the "A" line had proven its value as an important reference line, and that it should be kept essentially in its original position.

Casagrande suggested that in the expanded system a new group should be provided for the organic soils located over the "A" line. Also, in the old system, while the "A" line acted for the inorganic soils as a division between clays and silts, no such division extended to the organic soils. According to ASTM D2487-69, organic soils with a low plasticity could be classified as organic clays, ex: an organic soil with a liquid limit (LL) greater than 50 and a plasticity index (PI) less than 10. For such a soil, the term organic "clay" is not suitable, in view of its low plasticity.

For all these reasons the standards were changed in order that OL and OH soils could appear either above or below the "A" line. According to Howard (1984), the group name would depend on whether the soil plots above or below the 'A' line. The group name "organic clay" would apply to soils on or above the "A" line and "organic silt" would apply to soils below the "A" line. The liquid limit of 50 would remain the division between the symbols OL and OH, as shown in Fig. 2.1. Thus, the possible classifications for these soils were:

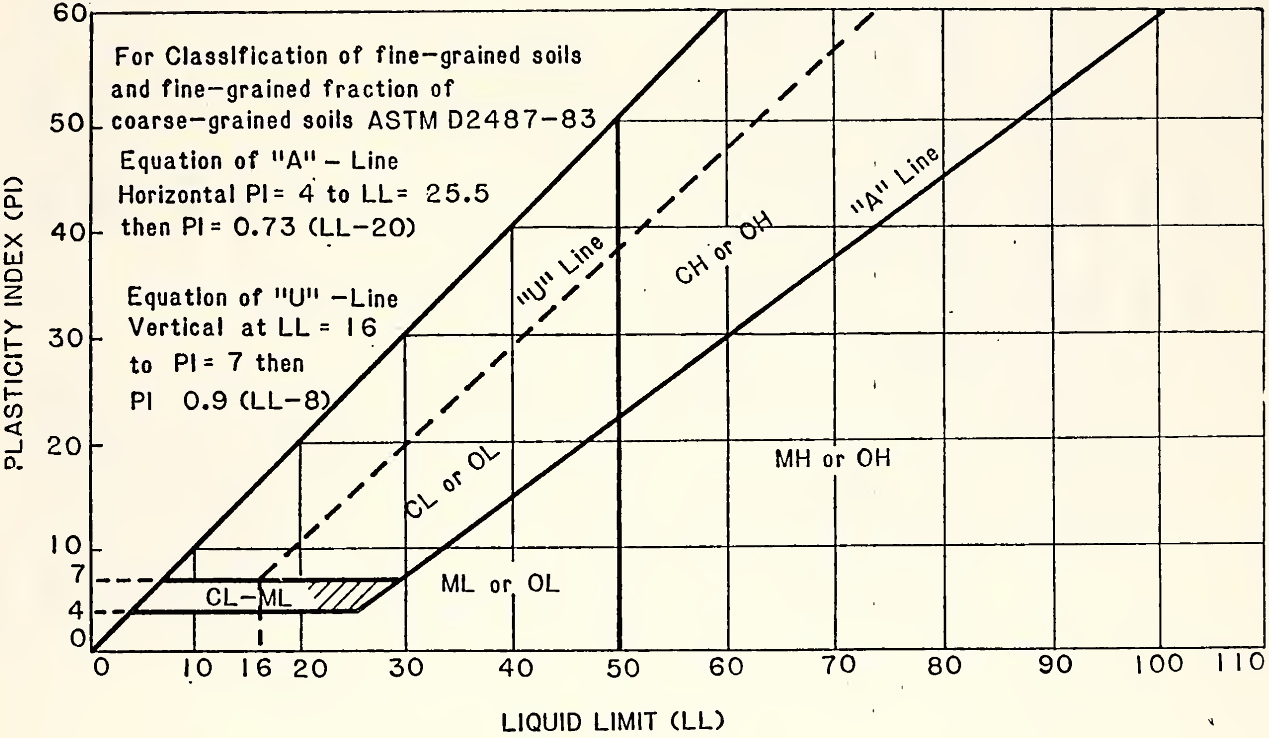


Figure 2.1 Plasticity Chart.

organic clay	OL
organic silt	OL
organic clay	OH
organic silt	OH

Fig. 2.2 shows the present classification chart (ASTM D2487-83) for use in classifying soils. Once a soil is classified as organic, the flow chart shown in Fig. 2.3 is used for determining the group name. Thus, for example, an "organic clay with gravel" is the group name for a soil whose liquid limit after oven drying is less than three-fourths of its liquid limit in the natural state (criterion for organic soil), whose liquid limit is less than fifty (criterion for OL or OH) whose plasticity index is greater than or equal to four (criterion regarding the "A" line), between 15 to 30 percent of which is greater than a No. 200 sieve, and in which the percentage of sand is less than the percentage of gravel. Details on how well the system works are not yet available.

It is worth noting that while the importance of the mineral fraction in influencing the behavior of a soil with an organic content exceeding 50 percent is probably minimal, as long as the organic content does not exceed seventy-five percent (not a peat), the material will be classified on the basis of its mineral content which has a minimal effect on the engineering properties.

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Soil Classification: Group Symbol	Soil Classification: Group Name[B]
Coarse-Grained Soils More than 50 % retained on No. 200 sieve	Gravels More than 50 % of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5 % fines[C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$[E]	GW	Well-graded gravel[F]
			$Cu < 4$ and/or $1 > Cc > 3$[E]	GP	Poorly graded gravel[F]
		Gravels with Fines More than 12 % fines[C]	Fines classify as ML or MH	GM	Silty gravel[F,G,H]
			Fines classify as CL or CH	GC	Clayey gravel[F,G,H]
	Sands 50 % or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5 % fines[D]	$Cu \geq 6$ and $1 \leq Cc \leq 3$[E]	SW	Well-graded sand[I]
			$Cu < 6$ and/or $1 > Cc > 3$[E]	SP	Poorly graded sand[I]
		Sands with Fines More than 12 % fines[D]	Fines classify as ML or MH	SM	Silty sand[G,H,I]
			Fines classify as CL or CH	SC	Clayey sand[G,H,I]
Fine-Grained Soils 50 % or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	inorganic	PI > 7 and plots on or above "A" line[J]	CL	Lean clay[K,L,M]
			PI < 4 or plots below "A" line[J]	ML	Silt[K,L,M]
		organic	$\frac{\text{Liquid limit - oven dried}}{\text{Liquid limit - not dried}} < 0.75$	OL	Organic clay[K,L,M,N] / Organic silt[K,L,M,O]
	Silts and Clays Liquid limit 50 or more	inorganic	PI plots on or above "A" line	CH	Fat clay[K,L,M]
			PI plots below "A" line	MH	Elastic silt[K,L,M]
		organic	$\frac{\text{Liquid limit - oven dried}}{\text{Liquid limit - not dried}} < 0.75$	OH	Organic clay[K,L,M,P] / Organic silt[K,L,M,Q]
Highly organic soils	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-in. (75-mm) sieve.
[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C] Gravels with 5 to 12 % fines require dual symbols:
GW-GM well-graded gravel with silt
GW-GC well-graded gravel with clay
GP-GM poorly graded gravel with silt
GP-GC poorly graded gravel with clay
[D] Sands with 5 to 12 % fines require dual symbols:
SW-SM well-graded sand with silt
SW-SC well-graded sand with clay
SP-SM poorly graded sand with silt
SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10}$ $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ≥ 15 % sand, add "with sand" to group name.
[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[H] If fines are organic, add "with organic fines" to group name.
[I] If soil contains ≥ 15 % gravel, add "with gravel" to group name.
[J] If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.
[K] If soil contains 15 to 29 % plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L] If soil contains ≥ 30 % plus No. 200, predominantly sand, add "sandy" to group name.
[M] If soil contains ≥ 30 % plus No. 200, predominantly gravel, add "gravelly" to group name.
[N] PI ≥ 4 and plots on or above "A" line.
[O] PI < 4 or plots below "A" line.
[P] PI plots on or above "A" line.
[Q] PI plots below "A" line.

Figure 2.2 Soil Classification Chart (ASTM D2487-83).

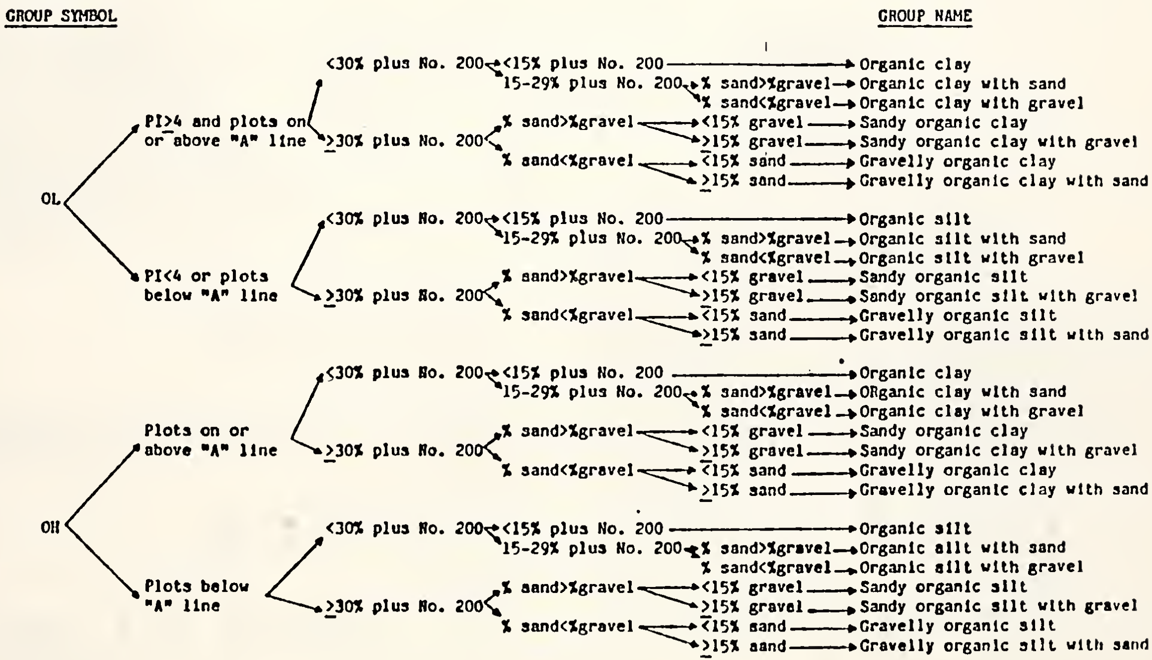


Figure 2.3 Flow Chart for Classifying Organic Soil (ASTM D2487-83).

CLASSIFICATION OF PEATS

If the organic content of the soil, as determined from an ashing test, lies between 75 to 100 percent, it is a peat. Fig. 2.4 shows a procedure recommended by Burwash and Wiesner (1983) to identify a peat. They recommend that, for geotechnical purposes, the classification be based upon the von Post Humification Scale. This is a visual-manual classification test. In this test a handful of material representative of the deposit is squeezed in the palm of the tester. Based on the color of the water, the amount of amorphous material squeezed out, etc., the material is classified as belonging to one of ten possible degrees of humification - H_1 to H_{10}. The classification can be accomplished in the field or the laboratory. It is a simple test once some initial experience is obtained. The method of identification is as shown in Table 2.1. More details can be found in the report by Gruen and Lovell (1983).

All peats whose degree of humification ranges from between H_1 to H_3 are described as fibrous. The fibers form a very visible component and are important in that they strongly influence the engineering behavior of the material.

This report deals with amorphous peats and mucks and therefore, all the information in it is specifically applicable only to such materials unless otherwise mentioned. Details on the fibrous peats can be found in the report by Gruen and Lovell (1983). Materials which lie in the range H_4 to H_{10} are described

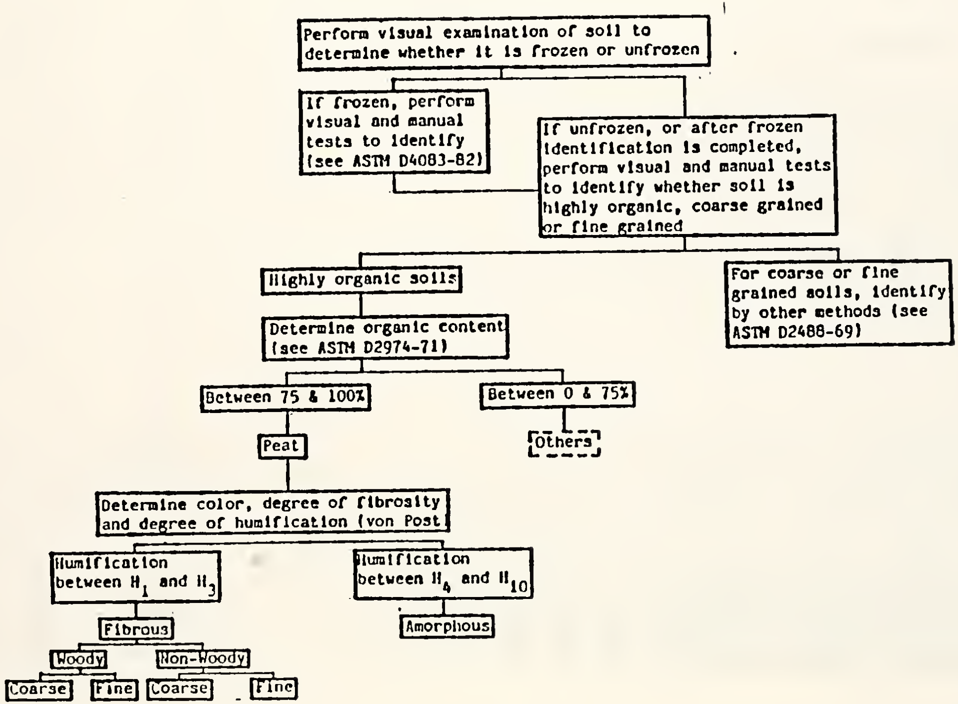


Figure 2.4 Peat Identification Procedure. From Burwash and Wiesner (1984).

Table 2.1 Determination of Humification (von Post 1922).

Degree of humification	Decomposition	Plant structure	Content of amorphous material	Material extruded on squeezing (passing between fingers)	Nature of residue
H_1	None	Easily identified	None	Clear, colorless water	
H_2	Insignificant	Easily identified	None	Yellowish water	
H_3	Very slight	Still identifiable	Slight	Brown, muddy water; no peat	Not pasty
H_4	Slight	Not easily identified	Some	Dark brown, muddy water; no peat	Somewhat pasty
H_5	Moderate	Recognizable, but vague	Considerable	Muddy water and some peat	Strongly pasty
H_6	Moderately strong	Indistinct (more distinct after squeezing)	Considerable	About one third of peat squeezed out; water dark brown	
H_7	Strong	Faintly recognizable	High	About one half of peat squeezed out; any water very dark brown	
H_8	Very strong	Very indistinct	High	About two thirds of peat squeezed out; also some pasty water	Plant tissue capable of resisting decomposition (roots, fibres)
H_9	Nearly complete	Almost not recognizable		Nearly all the peat squeezed out as a fairly uniform paste	
H_{10}	Complete	Not discernible		All the peat passes between the fingers; no free water visible	

as amorphous peats. The engineer thus has to distinguish between fibrous and amorphous peats, as these materials have different forms of observed creep behavior [Berry and Poskitt (1972)].

Fig. 2.5 shows the peat classification system as suggested by Burwash and Wiesner (1983). The group symbol for any peat is P_t, followed by the degree of humification in parenthesis, $P_t(H_6)$. The graphic symbols used for amorphous peats and fibrous peats are similar but differ enough to indicate two different materials. The naming procedure is straight forward, for example, black amorphous peat $P_t(H_8)$.

For field identification purposes a visual examination is sufficient. In the laboratory, the exact organic content, water content, etc. can be determined. Some researchers such as Landva et al (1983) and Cohen (1983) recommend the use of an electron microscope and oriented microtone sections, respectively, but such tests are not practical for engineering purposes.

One other system of classifying peat is the ASTM (1983) system which, from the results of a few laboratory tests, classifies peat on the basis of its fiber content, ash content, acidity, absorbency and botanical composition. Full details of this system are given in the report by Gruen and Lovell (1983). Landva (1980) has proposed a modified von Post system. The disadvantage of this system is that some botanical knowledge is required in order to classify the material into its genera.

				Major Divisions and Subdivisions	Group Symbols		Typical Names	Field Identification Procedures	Laboratory Classification Criteria
				Fibre Length	Letter	Graphic			
PEAT	Fibrous	Woody	Coarse	Short	Pt		Woody, coarse, fibrous	Visual	Water content
				Long	(H_1 to H_3)			Examinations	Ash content
			Fine	Short			Woody, fine, fibrous	of Color,	Detailed visual and manual tests for fibrosity, size and character of fibres, degree of humification
				Long				Fibrosity	
		Non-Woody	Coarse	Short			Non-woody, coarse, fibrous	& Degree of	
				Long				Humification	
			Fine	Short			Non-woody, fine fibrous		
				Long					
	Amorphous	H_4 → H_{10}		N/A	Pt (H_4 to H_{10})		Amorphous		

DEFINITIONS OF COMPONENTS

Fibrous - containing fibres which are well preserved and easily distinguishable.
Amorphous - highly decomposed (humification), with no evident structure.
Woody - containing remains of trees and shrubs.
Non-woody - absence of remains of trees and shrubs.
Coarse - fibres, stems and rootlets greater than 1 mm in diameter or width.
Fine - fibres, stems and rootlets smaller than 1 mm in diameter or width.
Short - fibres which are less than one-half of the sample width.
Long - fibres which are greater than one-half of the sample width.

Figure 2.5 Peat Classification and Description. From Burwash and Wiesner (1984).

ENGINEERING CORRELATIONS

Several previous investigators have tried to correlate the index properties with the engineering properties, for example, MacFarlane (1969), Radforth (1977) and Walmsley (1977). More success was achieved in forming correlations for organic soils than for peats. As mentioned by Burwash and Wiesner (1984) the reason for this is that most of the correlations were based on the water content calculated on the basis of dry weight. For peats, the water content can vary from 100 percent to as much as 2500%. This variation occurs seemingly both between peat types and even within a particular peat. Also, sampling and testing of these materials is not easy, and hence, experimental scatter is bound to be more than that for normal soils. For these two reasons, no straight forward correlation appears possible.

Keyser and Laforte (1984) found no clear relationship between the long term settlement and depth of peat deposit, von Post classification, shear strength of the material or the type of loading. Some authors, for example, Landva (1983), tried to relate the specific gravity to ash content, with a limited amount of success.

Burwash and Wiesner adopted the approach of correlating the peat type with certain engineering properties. In order to make these correlations, both field and laboratory data from New Brunswick and Manitoba were obtained. In all a total of about 7,000 data points were studied. These data points were plotted to determine the relation between water content and degree of humification. Among the fibrous peats, there was considerable scatter. This was attributed to difficulty in determining the true water content, as a significant amount of water escapes during the sampling process for fibrous materials. For 80 percent of their data it was found that the water content varied by ±150 percent for any humification value. Using this relation, ranges of moisture content were assigned to each humification value, through which it was possible to relate degree of humification to the engineering properties. Examples of the correlations found are shown in Fig. 2.6 to Fig. 2.8.

Landva and La Rochelle (1983) summarized a large number of tests and developed the relation shown in Fig. 2.6. As the fiber content increases, the compressibility increases. Fibrous peats, since they are relatively undecomposed, have a higher compressibility than the amorphous peats. In general, the compressive index of fibrous peats lies between 7 and 16. As humification increases, the compressibility decreases and therefore, the more humified amorphous peats can be expected to be less compressible with a compression index of between 3.0 and 7.0.

Fig. 2.7 shows the relation between shear strength and the peat type. These data are from numerous engineers and researchers. The shear strength used here is from the vane shear test. Fibrous peats have a relatively high shear strength. This

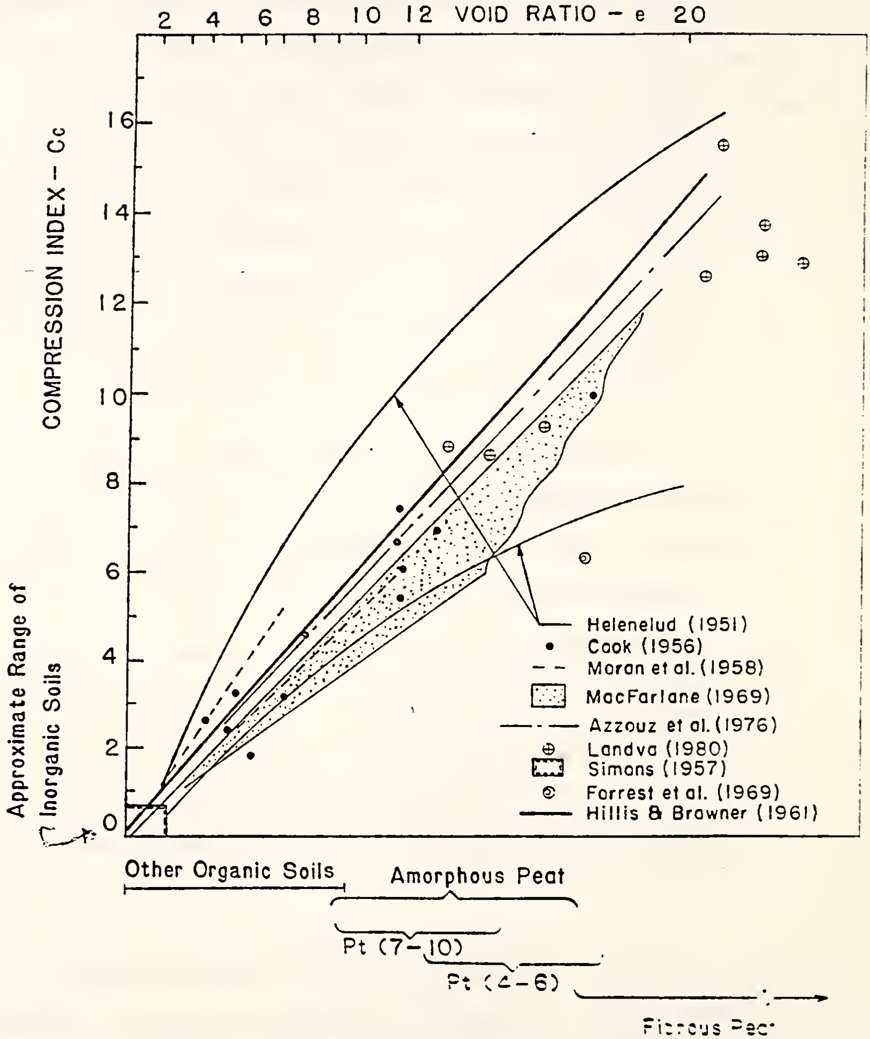


Figure 2.6 Peat Classification Related to Compression Index. From Landva and La Rochelle (1983c).

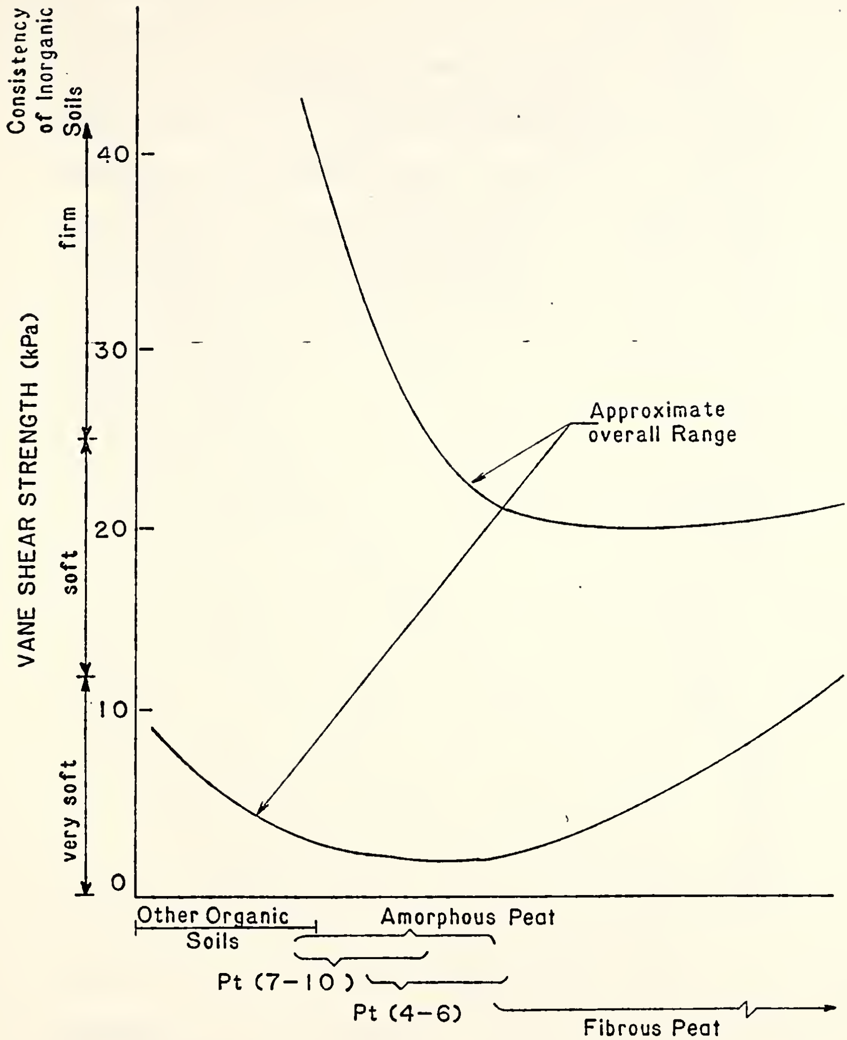


Figure 2.7 Peat Classification Related to Shear Strength. From Burwash and Wiesner (1984).

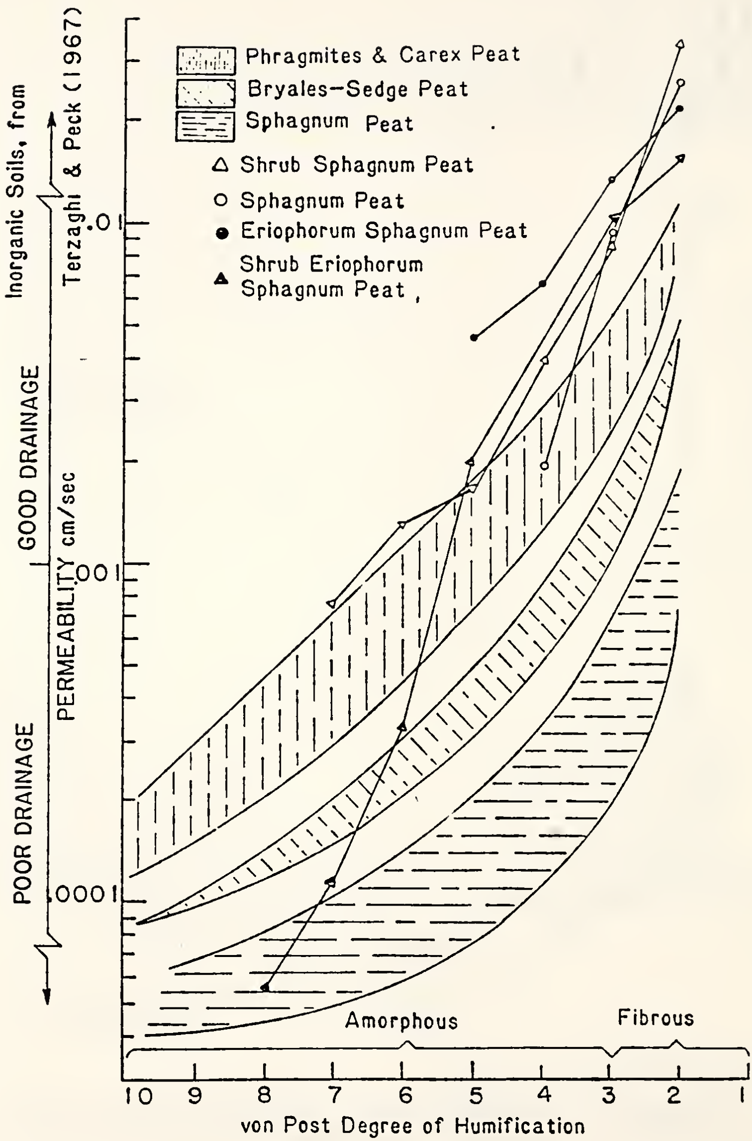


Figure 2.8 Peat Classification Related to Permeability. From Burwash and Wiesner (1984).

is because, in many cases, the vane gets entangled with the fibers which are relatively intact and undecomposed. As these soils exist at a high water content, such behavior is anomalous when compared with that of inorganic soils. As the degree of humification increases, it was expected that the vane strengths would increase because of the greater densification, relatively lower moisture content, and greater contamination by mineral soil. Within the range H_4 to H_6, the lowest shear strengths were obtained. Burwash and Wiesner (1983) attributed this to reduction of the shear strength of the fibers through decomposition, the relative low density, high moisture content, and also, a possible absence of any mineral contamination.

Good, Marsan and Michaud (1977) established a relationship between permeability and degree of humification. Burwash and Wiesner (1983) replotted their results using a semi-logarithmic scale, as shown in Fig. 2.8. The significant change in permeability with degree of humification is evident. Further, for each humification level, a considerable variation in permeability is clearly seen. The reason for this is that it is not easy to accurately measure the permeability of such soils. Compression of the soil occurs during sampling, sample preparation and actual testing. Since all these factors are not directly controlled by the experimenter, the effects vary from sample to sample. This variation is thought to result in the varying permeability. Since peats are highly compressible, this effect is observed to a greater degree than for other soils. In general, fibrous peats have a permeability lying between 10^{-4} cm/sec and 10^{-2} cm/sec, and possess good drainage characteristics. For amorphous peats, the permeability ranges between 10^{-3} cm/sec and 10^{-5} cm/sec, depending on the degree of humification.

The correlations between the type of peat and engineering properties presented above were obtained from data provided by several investigators. It is highly possible that apart from actual variation of material properties, the scatter could result from variations in classification from one person to another. Thus, one investigator may classify a peat as H_7 while another one may classify it as H_8 or H_6. Therefore, when properties are plotted as explained previously, it is only natural that there be a high scatter. A technique of classification using fuzzy sets offers considerable potential in eliminating this component of the variation.

A UNIFIED CLASSIFICATION SYSTEM

From the above discussion it is clear that there is no unified method of classifying peat, with the von Post system lacking important material available from the ASTM system and vice versa. The unified system suggested by the author is a combination of both systems, and is based on the system proposed by Landva (1980). The system, though oriented towards geotechnical engineering, may be of use to other fields.

I. (i) Fiber Content (by volume)

a) Fibric - Peat with more than $66\frac{2}{3}$ % fibers.

b) Hemic - Peat with between $33\frac{1}{3}$ and $66\frac{2}{3}$% fibers

c) Sapric - Peat with less than $33\frac{1}{3}$% fibers.

(ii) Fine Fibers (F): Less than 1 mm in width

F_0 Nil

F_1 Low Content

F_2 Moderate Content

F_3 High Content

(iii) Coarse Fibers (R)

R_0 Nil

R_1 Low Content

R_2 Moderate Content

R_3 High Content

(iv) Wood Content (W)

W_0 Nil

W_1 Low Content

W_2 Moderate Content

W_3 High Content

(v) Shrub Content (N)

N_0 Nil

N_1 Low Content

N_2 Moderate Content

N_3 High Content

II. Ash Content (ASTM Standard D2974) (by weight)

a) Low Ash Less than 5% ash

b) Medium Ash Between 5 and 15 % ash

c) High Ash Between 15 and 25 % ash

III. Acidity (ASTM Standard D2976)

a) Highly Acidic pH Less Than 4.5

b) Moderately Acidic pH Between 4.5 and 7.0

c) Basic pH Greater than 7.0

IV. Absorbency (ASTM Standard D2980)

a) Highly Absorbent Waterholding Capacity (WHC) Greater Than 1500 Percent

b) Moderately Absorbent WHC Lying Between 300 and 1500 Percent

c) Non-Absorbent WHC Greater Than 300 Percent

V. Botanical Composition (Floristic Designation)

Name Dominant Plants in the Fibers

VI. Water Content (B)

B_1 Less Than 100 Percent

B_2 Between 100 to 500 Percent

B_3 Between 500 to 1000 Percent

B_4 Between 1000 to 2000 Percent

B_5 Above 2000 Percent

VII. Humification (H)

This is done on the basis of the von Post scale previously mentioned and shown in Table 2.1.

USE OF FUZZY SETS FOR CLASSIFICATION

Fig. 2.6 to 2.8 show various correlations between the type of peat and various engineering properties. It is felt that a considerable part of this scatter arises from human uncertainty arising from various individual interpretations of the von Post humification scale. Thus, while one engineer may classify a peat as H_6, another may classify it as H_5 or H_7. Thus, any particular engineering property for this peat could be attributed by the engineer to be specific to H_5 peats or H_7 peats. This would result in considerable scatter for correlations of the type shown in Fig. 2.6 to Fig. 2.8. As seen, this scatter is a result not only of material variability, but also of human uncertainty. If this human uncertainty element is removed, the scatter can be reduced to that due to material uncertainty. Joseph and Lovell (1985) showed that fuzzy set theory offered considerable potential in reducing this human uncertainty to small values.

The original von Post humification scale was broken up into a standardized form as shown in Table C1. What distinguishes this table from the others is that each column contains only one descriptor such as "amount of amorphous material" or "amount of peat squeezed out". Further, the values of each descriptor have been standardized as far as possible. From the table it is clear that not all columns are required to uniquely define the degree of humification with, for example, only column II being sufficient. Hence, by deciding which columns are redundant, the number of membership functions that need to be defined may be considerably reduced. Further, it may not be necessary to use ten divisions for classification; fewer may be sufficient. Reducing the number of divisions from ten to a smaller value would further reduce the number of membership functions that would have to be developed.

The values of the linguistic variables as obtained by the field test are converted into fuzzy sets using the membership functions previously defined. These fuzzy sets are then matched with the rules. The matching can be done by calculating the distance between the appropriate fuzzy sets and finding the sum of the distances. The rule which best fits will have the smallest value of total distance. If further refinement is required, a weighted average can be used. Thus,

$$R_j = \left[\sum_{i=1}^{n} \frac{N_i}{D_i} \right]_j \qquad (2.1)$$

$$j = 1 \;\; 10$$

where N_i and D_i are the weights and distances corresponding to column i, the summation being done over ´n´, the number of columns considered significant. The R_j which has the highest value is the rule most applicable and so, the peat is classified as H_j, where ´j´ is the degree of humification.

The advantages of using fuzzy set theory as described above are:

1. No expert knowledge is needed to use the system correctly. Once the membership functions are defined, and guidelines given by experts, anyone can use the system accurately.

2. The system becomes more accommodating, as a peat whose description does not exactly correspond to any particular level of humification, can now be classified.

3. Weightage of the column descriptors is now possible, as previously explained. Thus, it is possible to quantitatively account for the relative importance of the column descriptors.

4. Because of these factors, the system is at the same time both more accurate and easier to use. It represents a standardization based on a framework that has been agreed upon by experts and so, a particular peat will always be classified at its true degree of humification, irrespective of the engineer classifying the peat, so long as the engineer involved uses the system as explained.

5. For the reasons described above, the element of human uncertainty will be removed, or made a very small component, resulting in better classification and hence, better correlations between engineering properties and peat type.

CONCLUSION

In conclusion, it must be emphasized that the correlations described above are to be used by the engineer only to gain a rough idea of the characteristics of the material being dealt with and should only be used as guidelines. No details are as yet available on how well the classification scheme for mucks works. A unified classification scheme for peats is suggested. Fuzzy set theory offers considerable potential in reducing the element of human uncertainty, resulting in better classification. Work is under way at Purdue University to investigate this potential. A preliminary questionnaire, used as a basis for the investigation, is shown in Appendix C.

CHAPTER III - SITE INVESTIGATION, SAMPLING AND SAMPLE PREPARATION

INTRODUCTION

The purpose of a site investigation is to obtain sufficient geotechnical information about the site, so that when the site is used for construction its behavior can be predicted. In a site investigation, two major controlling factors are the nature of the site and the nature of the project. Thus, if the proposed construction is a test embankment, a detailed and comprehensive testing program is required. If, on the other hand, the proposed construction is for ex: a low volume, low-cost road, no really detailed investigation is required. If the site is occupied by a highly variable soil like peat or muck, an elaborate investigation of single samples is seldom justified. Such an investigation will yield information about only a minute fraction of the total soil, and for a variable soil like peat or muck, it is not likely to provide information representative of the rest of the soil mass. Statistical methods too cannot be used, as the cost of obtaining sufficient information for such soils is prohibitive.

In many geotechnical investigations there is a disproportionate amount of care expended on testing, as compared to the amount spent during sampling. Terzaghi and Peck (1948) pointed out that the amount of testing and also the techniques used in testing were quite often out of proportion to the value of the results. Amorphous peats and mucks are soft materials with high compressibilities and low shear strengths. They are therefore particularly susceptible to disturbance during sampling, transportation and storage. Cooling (1949) emphasized that the validity of laboratory tests depends on the quality of the samples and the extent to which they are representative of the stratum.

Another factor to be considered is the soil underlying the peat. This soil too may be a problem material. Gruen and Lovell (1983) investigated the influence of marl beneath fibrous peats and concluded that the underlying soil layer may sometimes be the limiting factor.

Once samples are obtained, and brought to the laboratory, the question of preparing and installing the samples for testing arises. Methods such as extrusion, which work reasonably well for a large number of soils, considerably disturbs samples of peat and muck.

In view of these factors, it is clear that the testing of undisturbed samples in the laboratory is not easy, and requires specialized equipment. An extensive site investigation program is justified only for projects like test fills. This chapter presents information on site investigation, sampling and preparation of samples for testing, for soils such as peats and mucks.

SITE INVESTIGATION

As mentioned, the extent of a site investigation is strongly dependent on both the soil uniformity and the type of project. It is only for fairly uniform soil conditions, or for important projects, that an extensive investigation is warranted. For peats and mucks the most difficult part of the design study is to obtain representative values of the various engineering parameters. The average value, and the range of values are important, so that the design will take into account the worst possible conditions. For peats and mucks, it is preferable to carry out many simple and inexpensive tests from samples taken from over the area and within the depth of interest. These tests may be water content tests, ash content tests and, if possible, some oedometer tests and shear strength tests on reasonably undisturbed samples. Using data from these tests, and the correlations available, the area with the worst site conditions can be identified.

Once this area has been identified, undisturbed samples can be taken if required, and tested. For most projects on peats and mucks, an approach combining the predictive and observational methods of the type proposed by Gruen and Lovell (1983) is considered optimum. For such projects, once the critical area has been identified, tests conducted on block samples from at or near the surface, or from reasonably undisturbed samples from greater depths should do as a first approximation for input into the predictive part of the model.

On the other hand, if the project is a test fill and the approach basically predictive, great care will have to be taken in sampling and sample installation. The next two sections will outline a technique developed by Landva (1980), specifically for peats and mucks, that will ensure that the samples being tested are relatively undisturbed. In view of the complexity of the method, it is clear that it is applicable only where projects such as test fills, etc. are to be built.

For embankments, the depth of investigation should be at least equal to the width of the embankment. Ideally, it should penetrate all the soft soils below the embankment, if stability is to be investigated, and if possible, should extend into competent material.

At times, conventional trucks cannot be used in peat areas. In such situations, it may be necessary to use a tracked vehicle or a skid-mounted rig. The skid rig can be moved through the area by clearing obstructing vegetation, and winching the rig from location to location.

SAMPLING

The sampling method and the equipment to be used depends on the type of soil survey, i.e., whether it is a preliminary survey or a detailed survey. The object of a preliminary survey is to get a rough idea of the site conditions, with a minimum expenditure of time and effort. For peats and mucks a thorough preliminary survey will locate the areas with the worst conditions. Samples obtained from this sort of an investigation are known as "representative" samples, and may be obtained using plug samplers, hand operated augers, power augers and hand probes. Penetrometers can be used for obtaining initial data.

Swantko, et al (1984) used a probe of 5- to 10-foot sections of 0.5 inch diameter steel rods. They probed the area of interest on a grid pattern using a spacing of between 50 and 100 feet. They found that the probes could be pushed by hand through an organic layer, to a depth of about 20 feet without encountering much resistance. Beyond this point, they used wrenches, or a 40 pound drop hammer. Probing was not stopped at the first sign of increased resistance, as sometimes a sand layer would be interbedded in the peat or muck. The data from the probing can be used to develop an approximate contour map of organic thickness.

One such probe particularly suited for this sort of work and used often in the United Kingdom is the Mackintosh probe shown in Fig. 3.1. This probe is similar to that used by Swantko, et al

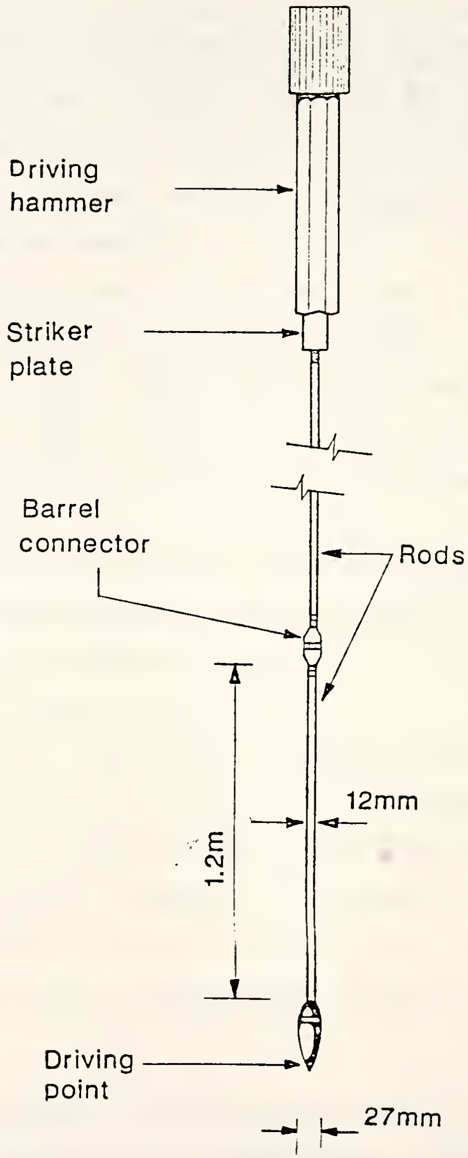


Figure 3.1 The Mackintosh Probe. From Clayton et al (1982).

(1984), except that the ends of each section are streamlined in longitudinal section. In the case of fairly stiff mucks, a depth profile can be obtained by driving the point and rods with equal blows of the hammer, using the full height of drop available. The number of blows required to advance six inches are counted. When a small layer or pocket of stiff clay is to be penetrated, an auger or a cone tube can be substituted for the driving point [Clayton, et al (1982)]. Using data from this preliminary survey, the critical areas are identified and a detailed exploration carried out in these areas.

Gordon (1985) classifies detailed explorations into several broad areas. Disturbed or drive sampling is accomplished using split wall samplers which produce samples which roughly look like the in-situ material. For peats and mucks such sampling will result in considerable disturbance. Broms (1980) stated that for peats an appreciable compression can occur during the sampling process, which can increase the shear strength of the material by up to 200%. This increase in shear strength is accompanied by a reduction in compressibility. Samples from this type of exploration can only be used to determine ash contents, fiber content, etc., in order to classify the material.

One problem that arises in any sampling is the removal of the sample from the tube. Extrusion by normal methods will considerably disturb the sample. This problem can be partially overcome by using a split barrel for sampling. One split barrel sampler suited for peats and mucks is the specific laboratory sampler. This sampler is similar to the standard split tube sampler except that it has an inner sectionalized liner. The sectionalized liner allows samples to be used for consolidation tests, without excessive trimming. The height of the brass ring sections that make up the liner can be cut to match laboratory equipment. In materials like peat and muck, the sampler may penetrate very easily into the soil. Sometimes, the self-weight of the sampler is sufficient. In such cases, the driller should use extreme caution to prevent overdriving and compressing the sample.

The term 'undisturbed sample' is misleading because in reality such a sample does not exist. Stress relief occurs when a sample is removed from the ground. However, by reconsolidating under K_o, or even approximately K_o conditions in the laboratory, much of this disturbance can be removed [Ladd and Foot (1974)]. In general, an "undisturbed" sample means a soil sample that is taken in such a manner that the changes in the physical and chemical properties are insignificant. Undisturbed samples are normally obtained for detailed and accurate laboratory tests. These samples are usually obtained by any of a number of thin-walled piston samplers. While this may be suitable for some mucks, for most mucks and peats this is not so, as extrusion of the sample from the tube and subsequent preparation for testing will result in considerable disturbance. Hillis and Brawner (1961), Hardy (1965), Hollingshead and Raymond (1971), Helenelund et al (1972), Landva (1980) and Gruen and Lovell (1983) all pointed out the difficulties involved in the undisturbed sampling of peat. Their remarks also apply to many of the mucks.

Landva et al (1982) summed up the difficulties as follows. First, there is a change in volume of the sample because the gas present expands due to the stress relief that occurs as the sample is removed from the ground. Second, there is a loss of moisture and an accompanying increase in shear strength and decrease in compressibility, as the sample is extruded. Lastly, forcing the soil sample into a container for testing will also cause disturbance.

Korpijaakko and Radforth (1972) and Korpijaakko and Woolnough (1977) modified the Swedish 50mm stationary piston peat sampler. This modified sampler was further developed into a 100mm diameter piston sampler that is suitable for the undisturbed sampling of very soft soils [Landva et al (1982)]. As shown in Fig. 3.2, the sampler has a plexiglas insert to hold the sample. The plexiglas is protected during the sampling action by a bronze tubing. The remainder of the tubing is 100mm in diameter and made of stainless steel. The cutting edge is stainless steel and has an angle of 30 degrees with the vertical. A double leather cup is used for the creation of maximum possible suction. The sampler can be equipped with a foil for retaining very soft samples. The plexiglas cylinder constitutes the middle third and it is the soil from this portion that is used for

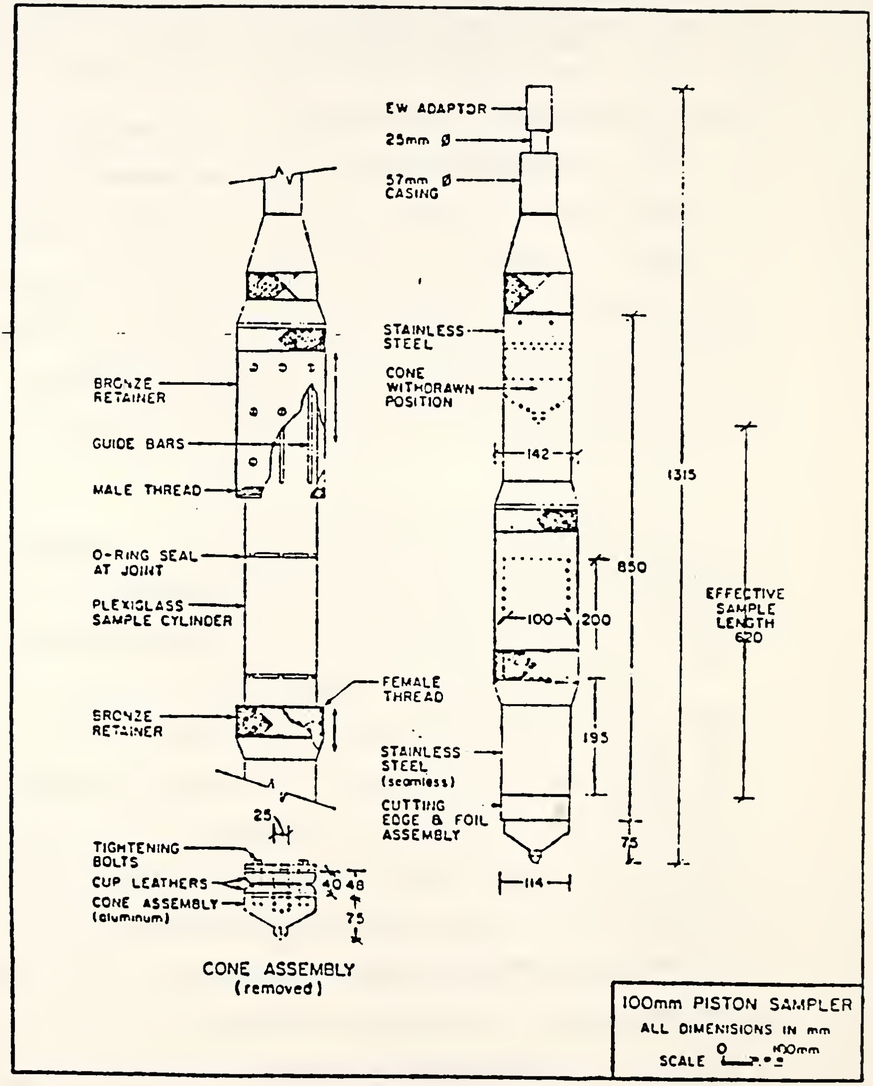


Figure 3.2 Landva's Piston Sampler. From Landva et al (1983a).

testing, the first and the last thirds being considered disturbed. The sampler has been reported by Landva et al (1982) as being capable of producing samples with minimal disturbance.

SAMPLE INSTALLATION

As peats and mucks are very soft, considerable disturbance results during extrusion and installation for testing, if normal procedures are used. Landva (1980) developed an effective method of preparing and mounting samples for testing. Landva's work was on fibrous peats and so to include the effects of fabric, he used a 100mm (4 in.) diameter sample. This required that the sampler and the testing equipment be capable of handling a sample of this diameter. The height of the sample was 200mm (8 in.). For amorphous peats and mucks, these dimensions can be reduced. Some peats and mucks are so soft they may be incapable of supporting their own weight when removed from the sampling tube. For such materials, undisturbed testing requires that the sample be constantly supported during installation.

The technique devised by Landva ensures that throughout the installation process, the sample is supported, and in order to minimize disturbance, at no time during the process is the sample touched by hand. This ensures that very soft organic materials can be installed with a minimum of disturbance. The sampling is done using the sampler previously described. The plexiglas cylinder is removed carefully and the undisturbed soil within it is sealed and taken to the laboratory. Once in the laboratory, the tube is displaced by a former with a rubber membrane. All movement is along guide rods so as to prevent any disturbance due to lateral movement. More details on installation of samples of peat can be found in Landva (1980).

A simpler method was followed during this consolidation and creep testing program. Block samples were taken from just below the surface in accordance with the procedures described by Winterkorn and Fang (1975). For oedometer testing, samples were obtained by pressing a ring into the soil. The ring was machined from stainless steel and had an outer diameter such that a slip fit was possible within the oedometer ring that normally held the soil. The wall thickness of the stainless steel ring was 0.020 inch. Hence, pressing the ring into the soil would cause minimum disturbance. The ring embedded in the soil was then scooped out, ensuring that considerable soil was present both beneath, above and to the sides of the ring. Now, the excess soil was carefully trimmed away and the oedometer ring carefully slipped around the thin-walled stainless steel ring. The sample was now ready for testing. It was felt that this method ensured the samples were undisturbed.

For the triaxial testing part of the program, however, this method was no longer possible and so, to ensure uniformity, remolded samples were used. Further details can be found in the chapter on testing.

CONCLUSION

Undisturbed sampling of peats and mucks is not easy, requiring special equipment. Such testing, however, is justified only for projects like test fills. For more routine projects, less rigorous testing procedures should be capable of providing at least some idea of the values of the geotechnical parameters of the most critical area. Once this preliminary idea is obtained, the procedures in Chapter Four can be used to estimate the magnitude of settlements likely to occur.

CHAPTER IV - THE COMPRESSION BEHAVIOR OF MUCKS AND AMORPHOUS PEATS

INTRODUCTION

Settlement predictions of embankments are normally based on the e-log p relation. This relation is approximated as linear for normally consolidated soils or as bi-linear for soils with a preconsolidation pressure. Examples of such relations are shown in Fig. 4.1. The rate at which settlement takes place is in most cases calculated on the basis of the Terzaghi theory. The exact details of the analysis can be found in the report by Goodman et al (1983). However, applications of similar theory to embankments on peat and other soils which have a highly nonlinear e-log p relation, and which exhibit significant secondary compression, have resulted in significant deviations between predicted and observed behaviors.

This discrepancy is attributed to factors such as the nonlinearity of the e-log p curve and secondary time effects, both of which are not accounted for in the conventional Terzaghi theory. Further, the Terzaghi theory is limited to those cases where loading results in strains that are small enough to be considered infinitesimal, and where the resulting change in

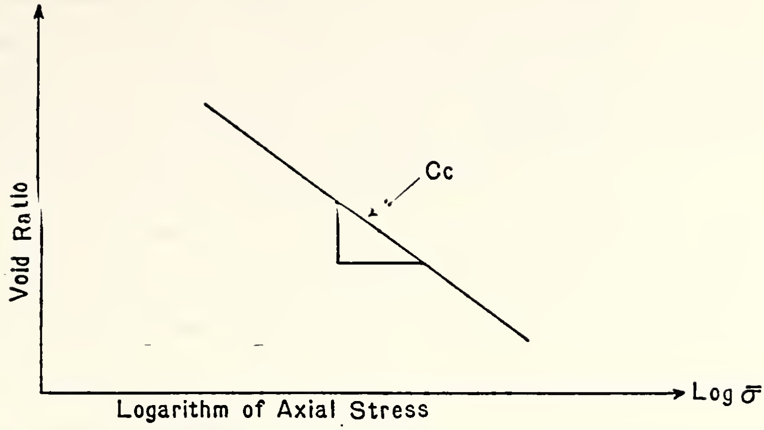


Figure 4.1a Linear e-log $\bar{\sigma}$ Relationship.

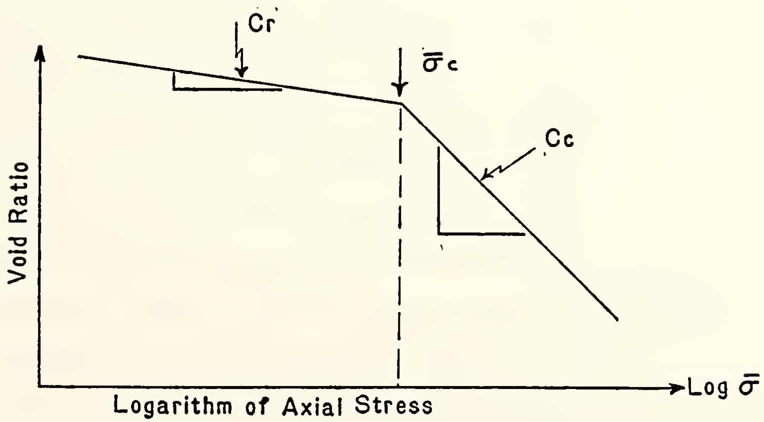


Figure 4.1b Bi-Linear e-log $\bar{\sigma}$ Relationship.

permeability is small enough to be considered negligible. Several authors have tried to extend the classical Terzaghi theory, to account for the variation of compressibility and permeability during consolidation, for ex: Richard (1957), Lo (1960), Davis and Raymond (1965), Janbu (1965), Barden and Berry (1965). However, all of these authors worked essentially within a framework of infinitesmal strain. This limitation was removed in the theory proposed by Gibson et al (1967). Schiffman (1980) showed that the conventional linear infinitesmal strain equation of Terzaghi, as well as various other nonlinear small strain theories, are special cases of the equation proposed by Gibson et al (1967).

As shown in Appendix A, the final form of the Gibson et al equation is for the one-dimensional consolidation of a material that is free from all the above limitations except that it has no intrinsic time dependency. Even a cursory study of available literature reveals that secondary compression is a major factor in the compression of peat, for ex: Berry and Poskitt (1972), Mesri (1973), Dhowian and Edil (1980) and Gruen and Lovell (1983). Mucks also exhibit significant secondary compression, but not much data are available, as they are often inaccurately classified as peats. Garlanger (1972) modified the classical Terzaghi equation using the soil model proposed by Bjerrum (1967), to account for secondary time effects. Using this approach he was fairly successful in modeling settlement behavior of the buildings at Drammen, Norway. However, his theory again has all the other limitations of the classical Terzaghi theory. Berry and Poskitt (1972) used the basic Gibson equation, which they modified to account for secondary compression. Later, Mesri and Rokhsar (1974) proposed a different modification of Gibson's equation to account for secondary compression.

The actual application of Berry and Poskitt's approach and Mesri's approach are reported first from a theoretical point of view, and then from a practical point of view. A simplified approach suggested by Mesri and Choi (1985), that accounts for nonlinearity, is proposed for calculating primary settlements. The choice of which method to actually use will depend on the nature of the problem, i.e., whether it is a test fill for research purposes, or a fairly routine engineering project. This is because the solution of the governing equations of Mesri, or of Berry and Poskitt, can only be solved numerically using a computer. On the other hand, the simplified approach requires only a calculator.

Finally, a theoretical analysis of the scaling to be used in predicting the field time for consolidation, based on laboratory time for consolidation, is presented.

ONE-DIMENSIONAL FINITE NONLINEAR CONSOLIDATION

All the major one-dimensional consolidation theories existing at present (1985) that apply to peats and to soils that exhibit secondary time effects are based on the equation due to

Gibson et al (1967). This equation takes into account nonlinearity of the e-log p curve, varying permeability, self-weight, and the finite nature of the strains. For this reason, a brief review of the theory is presented. The details are presented in Appendix B.

The basic assumptions of the theory (see Schiffman and Pane (1983)) are:

1. The saturated soil system consists of three components namely, soil particles, a soil skeleton and the pore fluid.

2. The soil particles and the pore fluid are incompressible.

3. The soil skeleton deforms in either a linear or nonlinear manner, with no restriction on the magnitude of strain.

4. The flow of fluid through the porous skeleton is governed by Darcy's Law.

5. The fluid flow velocities are small.

6. The pore fluid is Newtonian.

The theory is a quasi-static theory in that though movement of both the solid and liquid phase takes place, the inertial forces are assumed to be zero.

Within the framework described above, the following conditions are used:

1) Continuity of the solid phase and the liquid phase within the volume being considered. Thus, continuity of the solid phase requires that the net volumetric flux of solids across the boundaries of the volume being considered is equal to the time rate of change of solid volume within this element.

2) In addition to the principle of continuity, the properties of the material are to be specified. Thus, the void ratio should be specified as a single valued function of stress, independent of position and time. A similar single-valued relationship between the void ratio and the coefficient of permeability should also be specified. The single-valued nature of the function implies that the loading is monotonic. That the void ratio is independent of time implies that the soil is free from secondary time effects. The assumption that permeability and compressibility are a function of void ratio only implies that the soil layer is homogeneous.

3) The flow relationship followed by the fluid must be specified. The relationship used is Darcy's Law formulated in terms of the relative velocities between the solid and liquid phase.

4) Finally, the relationship between the total stresses applied to the soil and the actual stresses acting on the soil skeleton must be specified. This is done using the effective stress principle.

Based on the above, the most general one-dimensional, nonlinear finite strain consolidation theory was developed by Gibson et al (1967). The governing equation for this theory is:

$$\frac{\partial}{\partial z}\left[g(e)\frac{\partial e}{\partial z}\right] \pm f(e)\frac{\partial e}{\partial t} = \left(\frac{\partial e}{\partial t}\right)_p \qquad (4.1a)$$

where,

$$g(e) = -\frac{k(e)}{\gamma_w(1+e)}\frac{d\bar{\sigma}}{de} \qquad (4.1b)$$

$$f(e) = \left(\frac{\gamma_s}{\gamma_w} - 1\right)\frac{d}{de}\left[\frac{k(e)}{1+e}\right] \qquad (4.1c)$$

In which, e is the void ratio, γ_s and γ_w are the solid and fluid weights per unit of their own respective volumes, k is the coefficient of permeability, $\bar{\sigma}$ is the effective stress, and z is a reduced coordinate. This reduced coordinate is defined as:

$$z(a) = \int_0^a \frac{da'}{1+e(a_1,0)} \qquad (4.2)$$

where a is the Lagrangian (initial) coordinate point. Physically it is the volume of solids lying between the datum plane and the Lagrangian (initial) coordinate point. Use of a reduced coordinate system was suggested by Terzaghi (1927) and McNabb (1960). The upper/lower sign in Equation 4.1a is taken if the coordinate direction is measured against/with gravity.

Equations 4.1, together with the appropriate initial and boundary conditions, can be solved numerically for any sort of material property, measured or assumed, that satisfies the initial assumptions.

Since this is the basic equation which Berry and Poskitt (1972) and Mesri and Rokhsar (1974) extended and modified, it is worth commenting on. As pointed out by Schiffman and Pane (1983), it differs from the conventional theory in two major ways. First, it is not restricted in any way by the nature of the void ratio-effective stress and void ratio-permeability relationships. The relationships may be either linear or nonlinear. The precise form can be determined by laboratory tests. The theory, however, is restricted to the monotonic loading of homogeneous soils without any secondary time effects and without any chemical interactions between the constituents. Second, the theory is a finite strain theory and hence, is not restricted by the magnitude of the strains.

SECONDARY TIME EFFECTS

One reason for the deviations of predicted behavior from real behavior is that the theory on which most predictions are based do not account for secondary time effects, or if they do, they do so in a relatively simplistic way. In this section a brief review of secondary time effects is made. Two different approaches are presented. One is based on the rate-process theory and can be called a fundamental approach. The other is a more engineering type of approach. Each of these theories have been combined with the one-dimensional nonlinear finite strain consolidation equation and applied to peats.

At this point it should be noted that thus far the term "secondary time effects" has been used instead of terms like creep or secondary compression. The reason for this is that even a cursory study of case histories of construction on peat reveals that the terms ´creep´ and ´secondary compression´ have been used interchangeably. Lo (1961) and Crawford (1985) use the term ´secondary consolidation´ to describe settlements under constant effective stress. Leonards and Altschaeffl (1964) use the term ´creep´ for time dependent shear strains and the term ´secondary compression´ for time dependent volumetric changes, after excess pore pressures are essentially dissipated. From this point forward the term ´creep´ will be used for any deformation that occurs not as a result of an increase in stress but due only to microscopic interparticle movements. Secondary compression is considered to be drained one-dimensional creep. In general, it can be said that creep is the phenomenon, and secondary compression one of its manifestations.

The rate process theory was first formulated by Eyring (1936). Several investigators used this theory to describe the creep and consolidation behavior of soils. Rate process theory postulates that applied stresses are resisted by forces developed at interparticle contacts. These interparticle contacts are considered as being effectively solid to solid, regardless of whether two particles are in actual contact with one another, or whether they are separated by a layer of adsorbed water-cation complex. If such a layer exists, the force field existing at the interparticle near contact region is considered capable of generating an essentially solid interparticle region, which can transmit both shear and normal stress. Additional resistance to deformation exists as a particle being displaced must push adjacent particles out of its way as it moves to a new position, i.e., spaces must be formed into which a particle may move. This corresponds to a dilatency component, such as considered by Taylor (1948), Bishop (1953) and Rowe (1962). Mitchell (1964) states that resistance to interparticle movement is obtained from:

1. A frictional resistance generated by a normal force transmitted across the bond.

2. A stress independent cohesion dependent on physico-chemical forces of interaction and,

3. The effort that must be expended to cause dilation.

When in an unstressed state, particles occupy positions of minimum potential energy. For displacement to occur, energy must be supplied for the particle to surmount the potential energy barrier between equilibrium positions. The energy required to cause displacement is the sum of the energies required to cause failure at the contact and to form a hole to move into. The relation between the displacement from equilibrium and the energy required to cause the displacement is shown in Fig. 4.2. The energy required to reach the top of the barrier is called the activation energy, and will vary from contact to contact because of the variation of bond strengths existing from point to point. Let the mean activation energy be ΔE. If the mean interparticle normal force is P and the dilatency force is assumed proportional to P, then

$$\Delta E = \Delta E_o + P(\phi + D) \tag{4.3}$$

where, ΔE_o is the stress-independent physico-chemical component, ϕ is the stress dependent bond energy under a unit normal force, and D is the dilatency energy under a unit normal force.

Both the thermal energy in the soil mass and the shearing stresses applied to the soil are available as sources of energy to overcome the energy barrier. The average thermal energy of the interparticle contact zones is kT in which k is the Boltzmann constant and T the absolute temperature. The instantaneous value is continuously changing as the atoms oscillate about their equilibrium positions due to thermal vibrations. The energy distribution among all particles is defined by the Maxwell-Boltzmann equation which states that the probability of any energy state equal or greater than ΔE, is given by:

$$P(\Delta E) = (\text{const})\ e^{-\frac{\Delta E}{kT}} \tag{4.4}$$

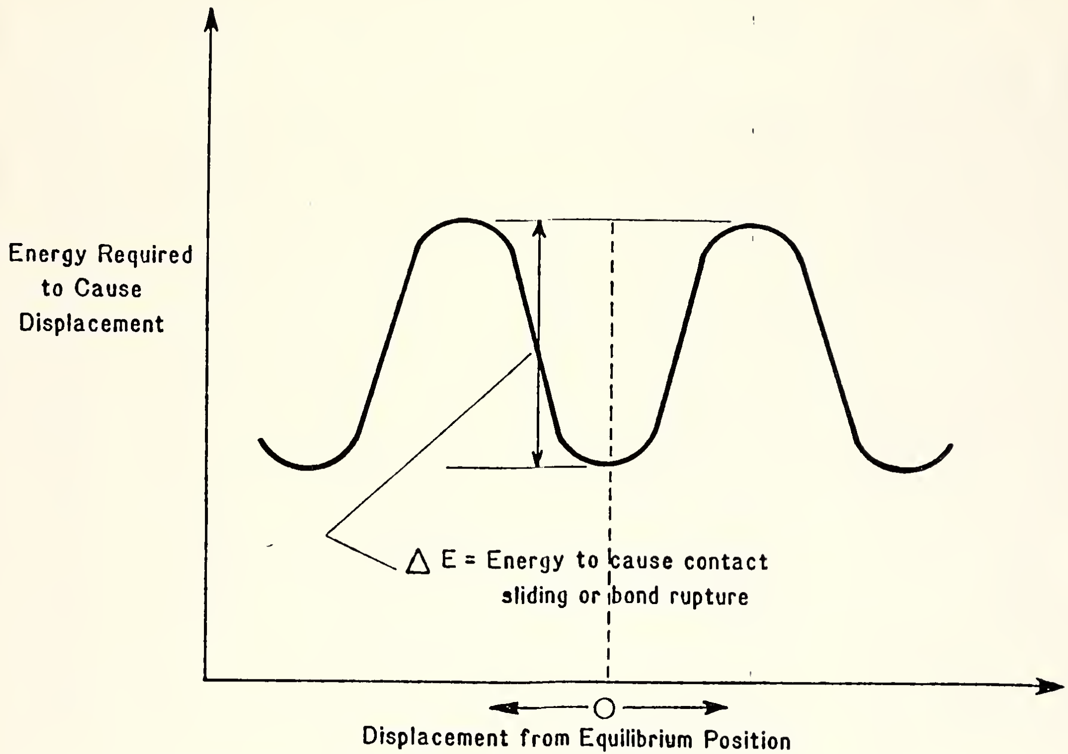


Figure 4.2 Energy Barriers Separating Successive Equilibrium Positions. From Mitchell (1964).

The quantity $e^{\frac{\Delta E}{kT}}$ represents at any instant the probability of any one interparticle bond being activated for rupture or the fraction of the total number of bonds that possess enough energy for rupture. Also, the value of the constant appears to be near unity, from the work of Glasstone, Laidler and Eyring (1941).

Glasstone et al (1941) also show that the mean frequency of thermal oscillations is kT/h where h is Planck's constant. Hence, the number of times per second, ν, that any bond acquires sufficient thermal free energy to overcome the energy barrier ΔE is:

$$\nu = \frac{kT}{h} e^{\frac{\Delta E}{kT}} \tag{4.5}$$

Activation and subsequent failure at a contact due to acquisition by a flow unit (an atom,, or a group of atoms, a molecule, or a group of molecules) in the contact zone of sufficient energy is assumed to initiate a progressive failure. Rupture occurs locally and propagates through the contact. Hence, the required energy input is not the sum of the energy barriers for displacement of all flow units in the contact zone. Confirmation comes from the fact that at effective stresses lower than that required to cause failure of the material, the creep rate has been known to increase until failure occurs (tertiary strain).

If there is no direction potential such as a shearing stress, bonds are activated equally in all directions and there is no tendency for particle movement in any one direction. If a shearing force f acts as shown in Fig. 4.3, a distortion of energy barriers results. A small instantaneous displacement of energy minima from A to A′ and from B to B′ occurs. As a flow unit oscillates along the slopes of the energy barrier, the force f works with the flow unit as it moves to the right and against the flow unit as it moves to the left.

The energy contributed by the force is $f\lambda/2$. If the barrier is symmetrical, and λ represents the distance between successful equilibrium positions, f results in a lowering of the potential barrier by an amount $f\lambda/2$ in the direction of the force, and an increase in the barrier by an amount $f\lambda/2$ in a direction opposite to the force. Hence, the frequency of activation to the right is:

$$\underset{\rightarrow}{\nu} = \frac{kT}{h} \exp\left[\frac{(\Delta E - \frac{f\lambda}{2})}{kT}\right] \qquad (4.6)$$

and to the left is:

$$\underset{\leftarrow}{\nu} = \frac{kT}{h} \exp\left[\frac{-(\Delta E + \frac{f\lambda}{2})}{kT}\right] \qquad (4.7)$$

The net frequency of displacements to the right is:

$$\underset{\rightarrow}{\nu} - \underset{\leftarrow}{\nu} = \frac{kT}{h} \left\{\exp\left[\frac{-(\Delta E - \frac{f\lambda}{2}}{kT}\right] - \exp\left[\frac{-(\Delta E + \frac{f\lambda}{2})}{kT}\right]\right\}$$

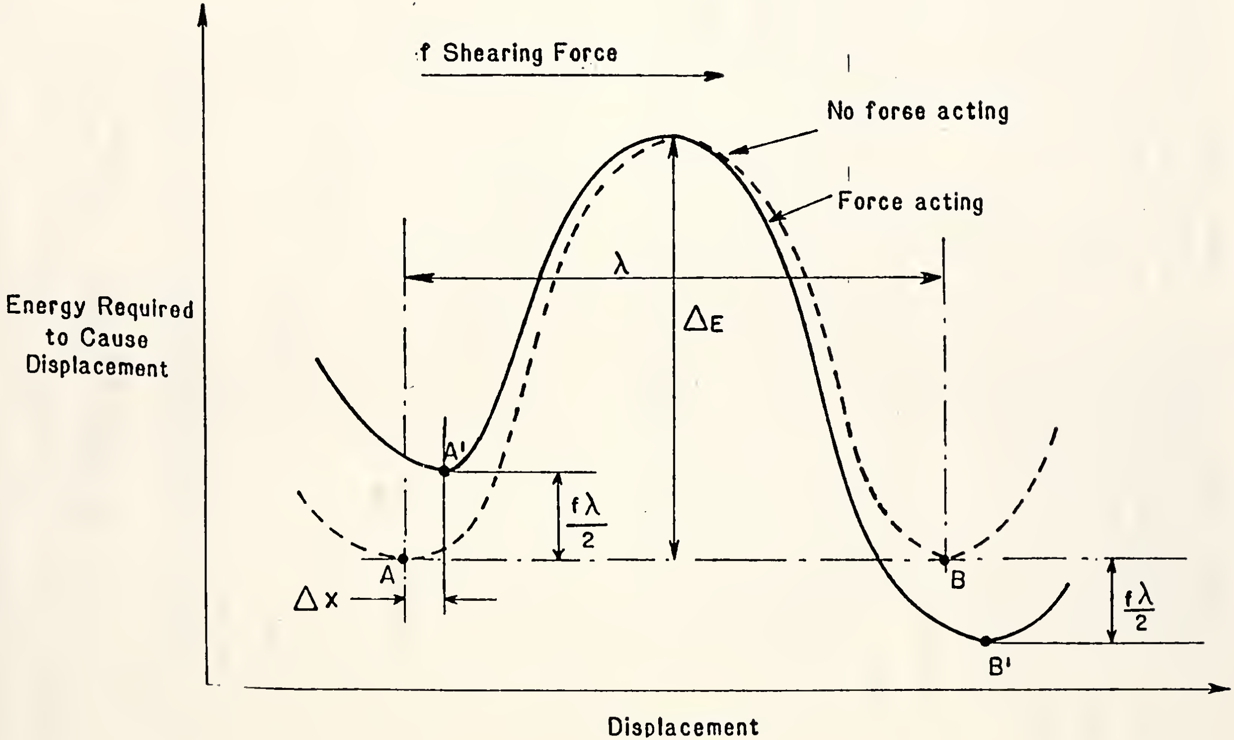


Figure 4.3 Influence of a Shearing Force on Energy Barriers Opposing Particle Movement. From Mitchell (1964).

$$\text{or } \underset{\rightarrow}{\nu} - \underset{\leftarrow}{\nu} = \frac{kT}{h} [e^{-\frac{\Delta E}{kT}} e^{+\frac{f\lambda}{2kT}} - e^{-\frac{\Delta E}{kT}} e^{-\frac{f\lambda}{2kT}}]$$

$$= \frac{kT}{h} e^{-\frac{\Delta E}{kT}} [e^{\frac{f\lambda}{2kT}} - e^{-\frac{f\lambda}{2kT}}]$$

$$= \frac{kT}{h} e^{-\frac{\Delta E}{kT}} [2 \sinh \frac{f\lambda}{2kT}]$$

$$= \frac{2kT}{h} e^{-\frac{\Delta E}{kT}} \sinh \frac{f\lambda}{2kT} \tag{4.8}$$

If we define x to be the displacement per unit length that occurs in the direction of deformation due to a single surmounting of the barrier, the strain rate is:

$$\dot{\varepsilon} = 2\, x \frac{kT}{h} \exp\left(-\frac{\Delta E}{kT}\right) \sinh\left(\frac{f\lambda}{2kT}\right) \tag{4.9}$$

Berry and Poskitt (1972) used the above equation to describe the secondary compression of amorphous peat. They approximated equation 4.9 to the form:

$$\dot{\varepsilon} = \beta(e) \sinh \alpha(e)\tau \tag{4.10}$$

$$\text{or } \frac{1}{1+e_o} \frac{de}{dt} = -\beta(e) \sinh \alpha(e)\tau \tag{4.11}$$

where the terms $\alpha(e)$ and $\beta(e)$ are rheological parameters which depend on the current void ratio. The values of these parameters decrease as the void ratio decreases.

The reason the author has presented the phenomenon of creep within the framework of a rate process is because, unlike the engineering-type theories of creep, the phenomenon is explained from first principles. The theory has been extended beyond that presented above by Fish (1983), but none of the extended theories have been combined with the consolidation equation.

Unlike the previous approach, the engineering approach does not make any assumptions about interparticle behavior, but tries to organize hitherto observed creep behavior into a coherent framework. One of the first studies of secondary time effects was by Buisman (1936), who said that settlements occur after dissipation of excess pore water pressure. There appears to be confusion in the literature as to which is the best way to describe the magnitude and rates of secondary settlement. Here, the definition of Raymond and Wahls (1976) and Mesri and Godlewski (1977) will be used. They define the secondary compression index C_α as:

$$C_\alpha = \frac{\Delta e}{\Delta \log t} \tag{4.12}$$

where Δe = the change in void ratio along a part of the e-log t curve between times t_1 and t_2 and $\Delta \log t$ = the difference between $\log t_2$ and $\log t_1$, and t_1, t_2 are two instants of time after primary consolidation is over.

According to Mesri ([illegible]), secondary compression may be defined as the continuation of the mechanism of volume change initiated during either instantaneous or primary compression. Mesri and Godlewski (1977) state that there is no logical reason why there should be differences between the mechanism of volume change during the changes in external effective stress and those redistributing the internal interactions. All the volume change mechanisms such as deformation, displacement, and aggregation or dispersion of particles, distortion of adsorbed water, and contraction and expansion of double layers can operate during the change in effective stress or with time. Further, they state that the magnitude of the changes in fabric and in the forces between the particles during an increment of time or with an increment of effective stress should depend on the previous changes with time and effective stress.

Lo (1961) related the typical e-log t curves shown in Fig. 4.4 with the load increment ratio. Others compared, at a given load increment, the magnitude of secondary settlement at any instant with the magnitude of total primary settlement.

Various authors such as Ladd (1971), Mesri (1973), Mesri et al (1975), Walker (1964), Walker and Raymond (1968, 1969) found various relationships between the compressibility with respect to effective stress and compressibility with respect to time. A typical form of the relationship is shown in Fig. 4.5, plotted in e-log t - log $\bar{\sigma}$ space. As a result of their work, Mesri and Godlewski (1977) postulate that there exists a unique relation between $C_\alpha = \dfrac{\partial e}{\partial \log t}$ and $C_c = \dfrac{\partial e}{\partial \log \bar{\sigma}}$ that holds true for all

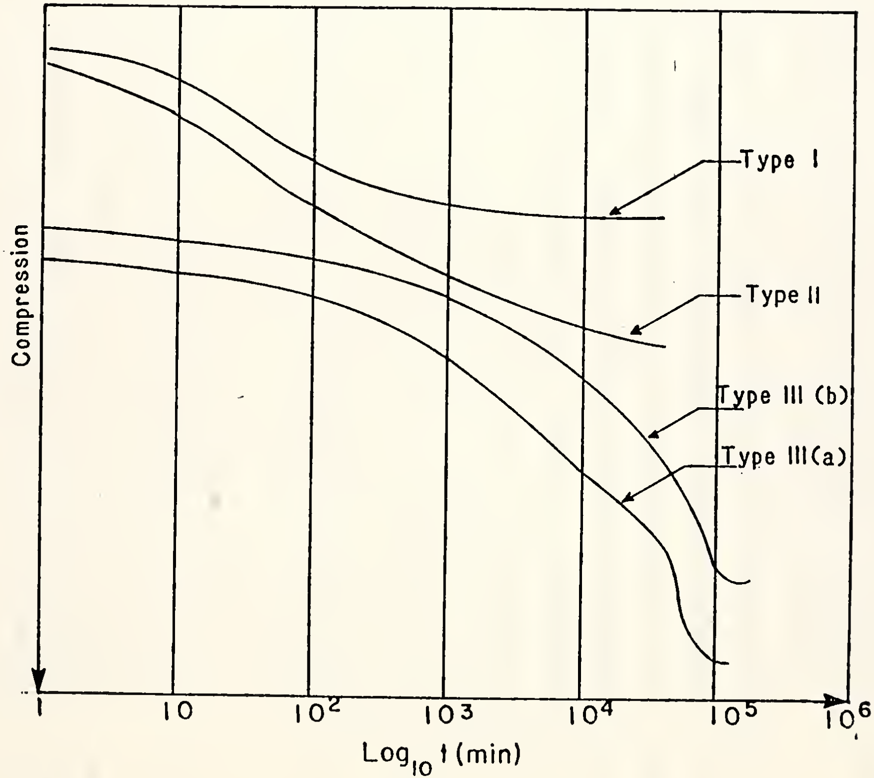


Figure 4.4 Types of Secondary Compression Curves. From Lo (1961).

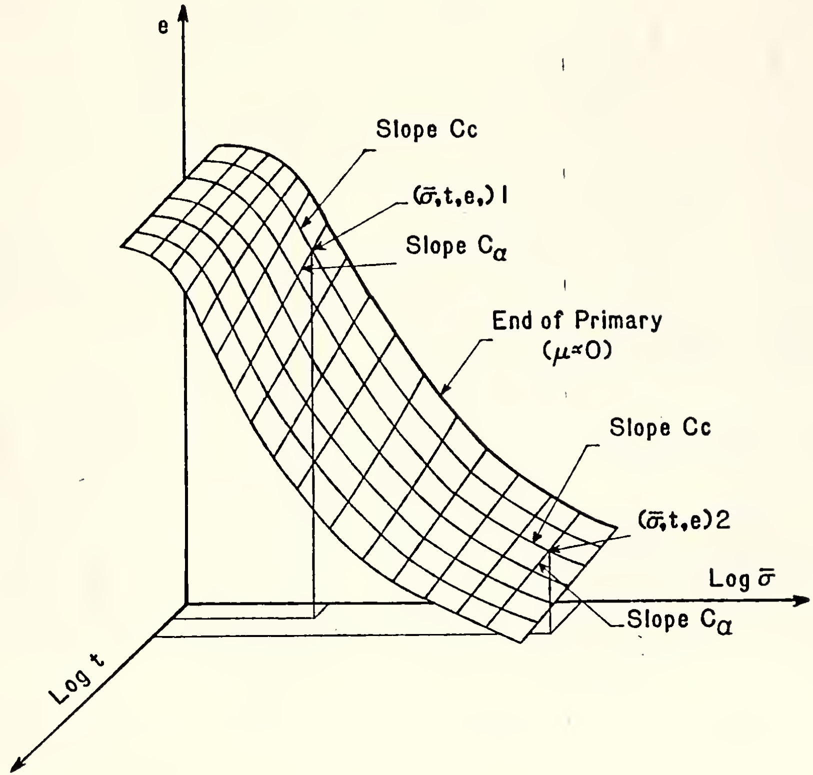


Figure 4.5 e-log $\bar{\sigma}$ - log t Plot Showing Relation between C_α and C_c During Secondary Compression. From Mesri and Godlewski (1977).

combinations of time, effective stress and void ratio, i.e., the ratio of the sides of any quadrilateral in Fig. 4.5 is a constant. Data on the C_α/C_c values for several soils as collected from a literature review by Mesri and Godlewski (1977) are shown in Table 4.1.

The implication of the postulate of a unique C_α/C_c ratio is that once the unique value of this ratio is determined, one can determine the changes of both C_α and C_c with time.

The procedure is to run step load tests so that the end-of-primary e-log $\bar{\sigma}$ curve can be defined. Next the relation between C_α and C_c is obtained, either as a unique value or as a plot such as in Fig. 4.6. Knowing the end-of-primary e-log $\bar{\sigma}$ relation and the C_α/C_c relation, it is possible to draw the e-log $\bar{\sigma}$ curve at 10 t_p, where t_p is the time required for primary consolidation. C_α is assumed to be constant between t_p and 10 t_p. Several points on the end-of-primary e-log $\bar{\sigma}$ curve are selected and C_c is determined at each of these points. Knowing C_c and the C_α/C_c ratio, C_α is found at each point, from which the change in void ratio at each point over 10 t_p (one log cycle of time) at that particular stress is determined, and the new point plotted. The same procedure is repeated for the other points and hence the e-log $\bar{\sigma}$ curve at 10 t_p is found.

Using this new curve and the same procedure, the e-log $\bar{\sigma}$ curve for 100 t_p can be found. A series of such curves is shown in Fig. 4.7. Mesri and Godlewski (1977) showed that in general,

Table 4.1 Values of C_α/C_c for Natural Soil Deposits.

Soil (1)	C_α/C_c (2)	Reference (3)
Whangamarino clay	0.03-0.04	Newland and Allely (1960)
Norfolk organic silt	0.05	Barber (1961)
Calcareous organic silt	0.035-0.06	Wahls (1962)
Amorphous and fibrous peats	0.035-0.083	Lea and Brawner (1963)
Canadian muskeg	0.09-0.10	Adams (1965)
Leda clay	0.03-0.055	Walker and Raymond (1968)
Leda clay	0.04-0.06	Walker and Raymond (1969)
Peat	0.075-0.085	Weber (1969)
Post-glacial organic clay	0.05-0.07	Chang (1969)
Soft blue clay	0.026	Crawford and Sutherland (1971)
Organic clays and silts	0.04-0.06	Ladd (1971)
Sensitive clay, Portland	0.025-0.055	Ladd (1971)
Peat	0.05-0.08	Samson and La Rochelle (1972)
San Francisco Bay mud	0.04-0.06	Su and Prysock (1972)
New Liskeard varved clay	0.03-0.06	Quigley and Ogunbadejo (1972)
Silty clay C	0.032	Samson and Garneau (1973)
Nearshore clays and silts	0.055-0.075	Brown and Rashid (1975)
Fibrous peat	0.06-0.085	Berry and Vickers (1975)
Mexico City clay	0.03-0.035	Mesri, et al (1975)
Leda clay	0.025-0.04	Mesri, et al (1977)
New Haven organic clay silt	0.04-0.075	Mesri, et al (1977)

From Mesri and Godlewski (1977)

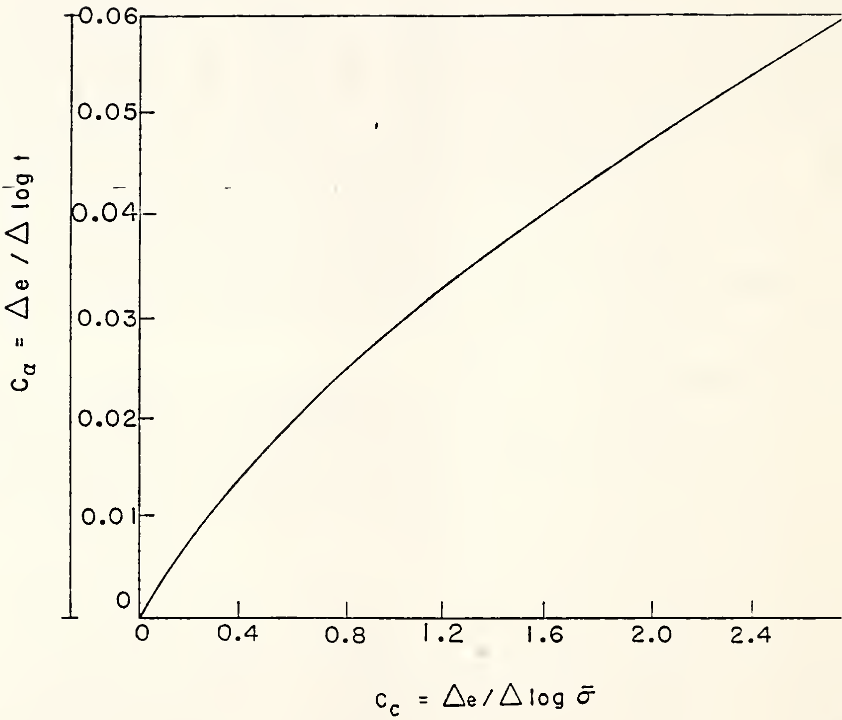


Figure 4.6 Relation Between C_α and C_c. From Mesri and Godlewski (1977).

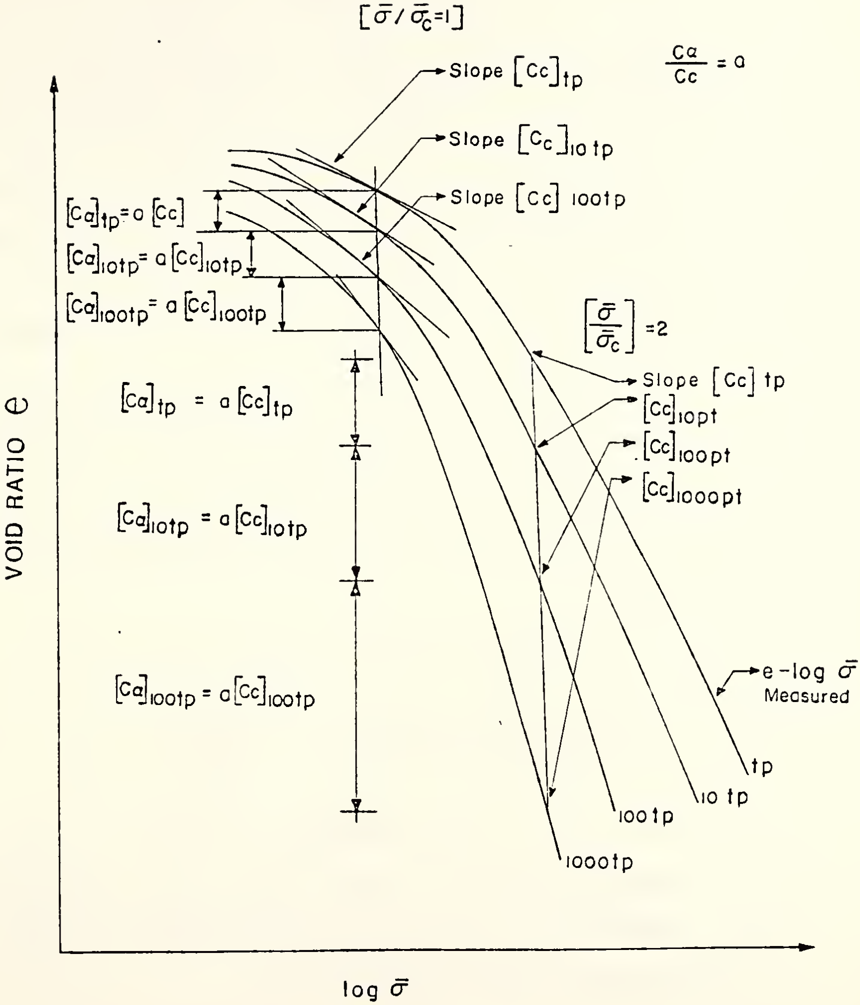


Figure 4.7 Procedure to Calculate the e-log $\bar{\sigma}$ Curves at Various Times. From Mesri and Godlewski (1977).

for stresses below the critical pressure, C_α first increases with time, levels off, and then decreases. The time at which it levels off increases as $\bar{\sigma}/\bar{\sigma}_c$ decreases. For pressures greater than the critical pressure, C_α decreases with time. For pressures near the critical pressure, C_α initially increases and then decreases. However, it must be mentioned that they could show this only by assuming that C_α/C_c had a unique value independent of time, void ratio or stress. Using this assumption, they extrapolated other e-log $\bar{\sigma}$ curves from the end-of-primary e-log p curve and hence, studied the variation of C_α.

All of the values of C_α/C_c lie between 0.025 and 0.10, the more organic soils having the higher values. Using these data it is now possible to investigate the relation between the load increment ratio and the shape of the settlement-logarithm of time curves.

The Terzaghi compression-logarithm of time curve has an inflection point at which the sense of concavity changes. The slope at this point is [Mesri and Godlewski (1977)]:

$$\frac{\partial e}{\partial \log t} = 0.7\ \Delta e_p \tag{4.13}$$

where Δe_p is the void ratio change up to the end-of-primary consolidation. When the initial effective stress $\bar{\sigma}_i$ is greater than the critical pressure $\bar{\sigma}_c$ then,

$$\frac{\partial e}{\partial \log t} = 0.7\ C_c \log\left(1 + \frac{\Delta\sigma}{\bar{\sigma}_i}\right) \tag{4.14}$$

If the load increment ratio is unity, from Equation 4.14,

$$\frac{\dfrac{\partial e}{\partial \log t}}{C_c} = 0.21$$

The maximum observed value of C_α/C_c is 0.10. Hence, for the load increment ratio of 1, there will be a large change in $\frac{\partial e}{\partial \log t}$, as primary consolidation is accomplished. In other words, for all soils, regardless of whether they are peats, mucks or clays, there will be a large change in $\partial e/\partial \log t$ as primary consolidation is accomplished, if $\bar{\sigma}_i > \bar{\sigma}_c$ and $\Delta\sigma/\bar{\sigma}_i = 1$. If, however, $\Delta\sigma/\bar{\sigma}_i$ is say, 0.2, then $(\partial e/\partial \log t)/C_c = 0.056$. If $C_\alpha/C_c > 0.056$, secondary compression will begin to control the e-log t curve as primary compression is completed, and there will be not much change in the slope to show that primary consolidation has been accomplished.

For the load increment straddling the critical pressure $\bar{\sigma}_c$,

$$\frac{\partial e}{\partial \log t} = 0.7\ C_c\ \left(\frac{C_c}{C_r} \log \frac{\bar{\sigma}_c}{\bar{\sigma}_i} + \log \frac{\bar{\sigma}_f}{\bar{\sigma}_c}\right) \qquad (4.15)$$

where C_r is the compression index and $\bar{\sigma}_f$ is the final effective pressure. The value of $(\partial e/\partial \log t)/C_c$ now depends on C_r, the pressure increment ratio $\Delta\sigma/\bar{\sigma}_c$, and also $\bar{\sigma}_c/\bar{\sigma}_i$. In other words, depending on the values of the variables involved, practically any shape can be obtained, i.e., the load increment ratio per se is not the controlling factor. However, it must be noted that tertiary creep strain occurs only for undisturbed soils, and only for the load increment straddling the critical pressure or for stresses near the critical stress.

C_α has been compared with the magnitude of primary settlement. However, while primary settlement increases with load increment, C_α may (as has been discussed) increase, remain constant or decrease.

Finally, the magnitude of secondary settlement at any increment of time has been compared with the primary settlement for a given load increment. The time for consolidation for a given load increment will vary depending on the variation of the coefficient of consolidation with stress level and the length of the drainage path. At any given time, the amount of secondary compression will depend on the duration of secondary compression and hence, on the duration of primary compression, which in turn is a function of the coefficient of consolidation or, the stress level. Hence, such a comparison is misleading.

However, laboratory evidence will be presented later, for tests run on certain organic soils and peats. For these soils, consolidation occurs so rapidly that the effect on long term creep curves is minimal. As a consequence, secondary compressions can be compared with the initial settlement.

This engineering approach to creep differs from the rate process theory in that it does not originate from the phenomena causing creep. Because of this, the author believes that the rate process theory is basically the direction to be followed in creep studies. Fish (1983) has combined a thermodynamic theory of the strength of solids and the rate process theory, to formulate a model that describes the entire creep process for constant stress and constant strain rate tests. However, this theory has not, so far, been combined with consolidation theory.

The occurrence of a sudden increase in the creep rate ultimately resulting in failure is called tertiary creep. When stresses are near failure levels, strains too are near failure levels. The resistance of a structure to creep will decrease as the cumulative strain (including creep strain) approaches the failure strain. This decrease in resistance results in an increased creep rate, and ultimately, when the cumulative strain reaches the failure strain corresponding to that stress level, failure occurs. Why tertiary strain occurs for stresses straddling the critical pressure is not clear at present. For peats and mucks, tertiary strain has been observed in the laboratory (Gruen and Lovell (1983)), but not in the field. This is because stresses in the field are beyond the critical stress level. Because of construction time and the speed at which consolidation occurs, the stress increment in the field cannot be considered as straddling the critical pressure.

THE BERRY-POSKITT THEORY

By neglecting the self weight of the layer, i.e. by putting $\gamma_s = \gamma_w$ in Equation 4.1c, Equation 4.1a reduces to

$$\frac{\partial}{\partial z}\left[\frac{k(e)}{\gamma_w\,(1+e)}\frac{\partial u}{\partial z}\right] = \frac{\partial e}{\partial t} \qquad (4.16)$$

By introducing certain material characteristics that seemed to apply to peat, Berry and Poskitt modified Equation 4.16 to describe the consolidation process for peat. To account for the large changes in permeability that occur in peats and other organic soils, they introduced a permeability law. They assumed a linear relation between the void ratio and the logarithm of the coefficient of permeability. Hence, from Fig. 4.8 it can be seen that:

$$\frac{e_o - e_f}{\log_{10} \frac{k_f}{k_o}} = \frac{e_o - e}{\log_{10} \frac{k}{k_o}}$$

$$\text{or } \log_{10} \frac{k}{k_o} = \frac{e_o - e}{e_o - e_f} \log_{10} \frac{k_f}{k_o} = \log_{10} \left(\frac{k_f}{k_o}\right)^{\frac{e_o - e}{e_o - e_f}}$$

$$\text{or } \frac{k}{k_o} = \left(\frac{k_f}{k_o}\right)^{\frac{e_o - e}{e_o - e_f}}$$

$$\text{or } k = k_o \left(\frac{k_f}{k_o}\right)^{\frac{e_o - e}{e_o - e_f}} \qquad (4.17)$$

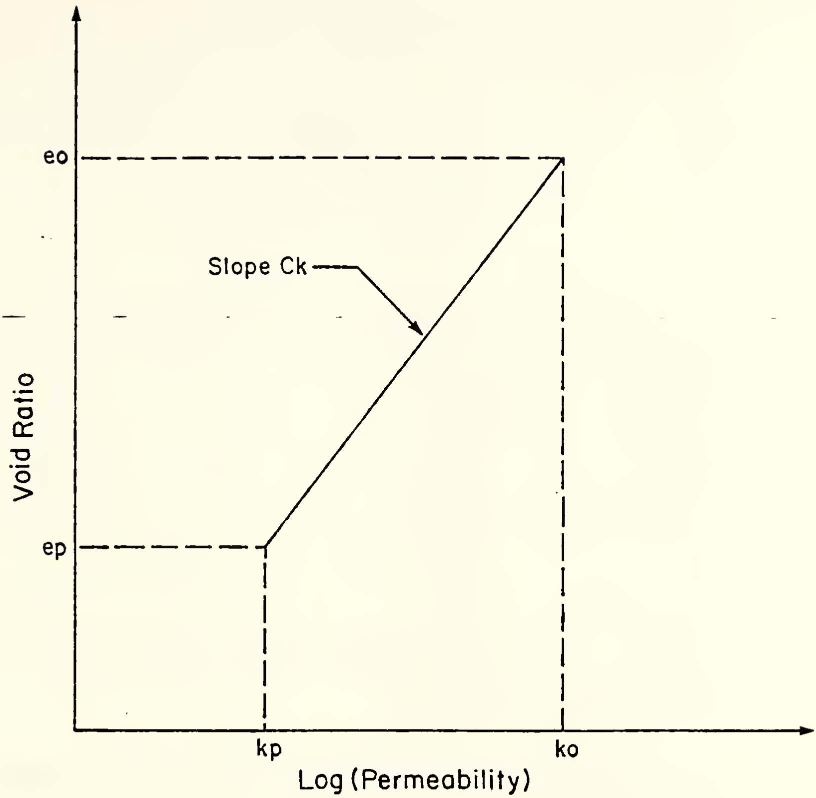


Figure 4.8 Idealized Relationship Between Void Ratio and Permeability.

Hence, equation 4.16 becomes

$$\frac{k_o}{\gamma_w}\frac{\partial}{\partial z}\left[\frac{\left(\frac{k_f}{k_o}\right)^{\frac{e_o-e}{e_o-e_f}}}{1+e}\frac{\partial u}{\partial z}\right] = \frac{\partial e}{\partial t} \tag{4.18}$$

Equation 4.18 is subject to the assumptions involved in equations 4-1, and so cannot account for creep. Berry and Poskitt assumed the rheological model shown in Fig. 4.9, which is essentially the same as that used by Gibson and Lo (1968), except the springs are nonlinear and the dashpot is non-Newtonian. This means that the proportionality factor between stress and strain rate is not a constant for the dashpot, and the stress-strain relation for the springs is not linear. This is done to simulate the nonlinear soil behavior.

Further, to indicate that creep occurs during the consolidation process, they placed this model into a Terzaghi pot (Fig. 4.10). However, they assumed the nonlinear e-$\bar{\sigma}$ relation to be linear in e-log $\bar{\sigma}$ space. The nonlinear viscous behavior of the soil was simulated with the dashpot, the non-Newtonian form of the relationship between stress τ and the strain rate de/dt being as shown in Fig. 4.11. This is a function of the void ratio. As the void ratio decreases, the viscosity of the soil increases and a given value of τ will produce a smaller value of de/dt.

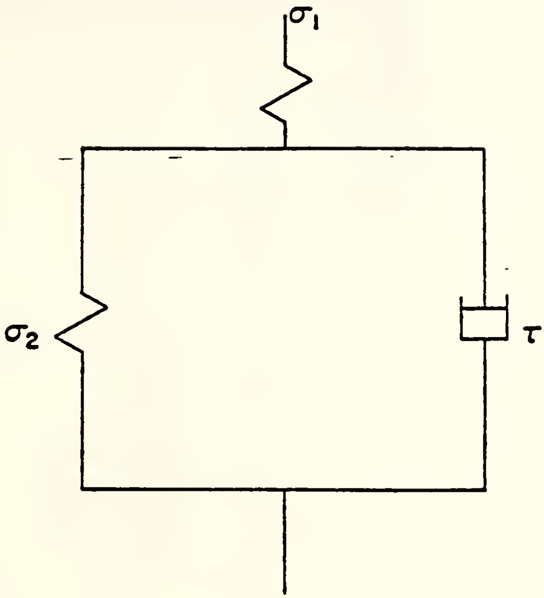


Figure 4.9 Berry and Poskitt's (1972) Model of the Soil Skeleton.

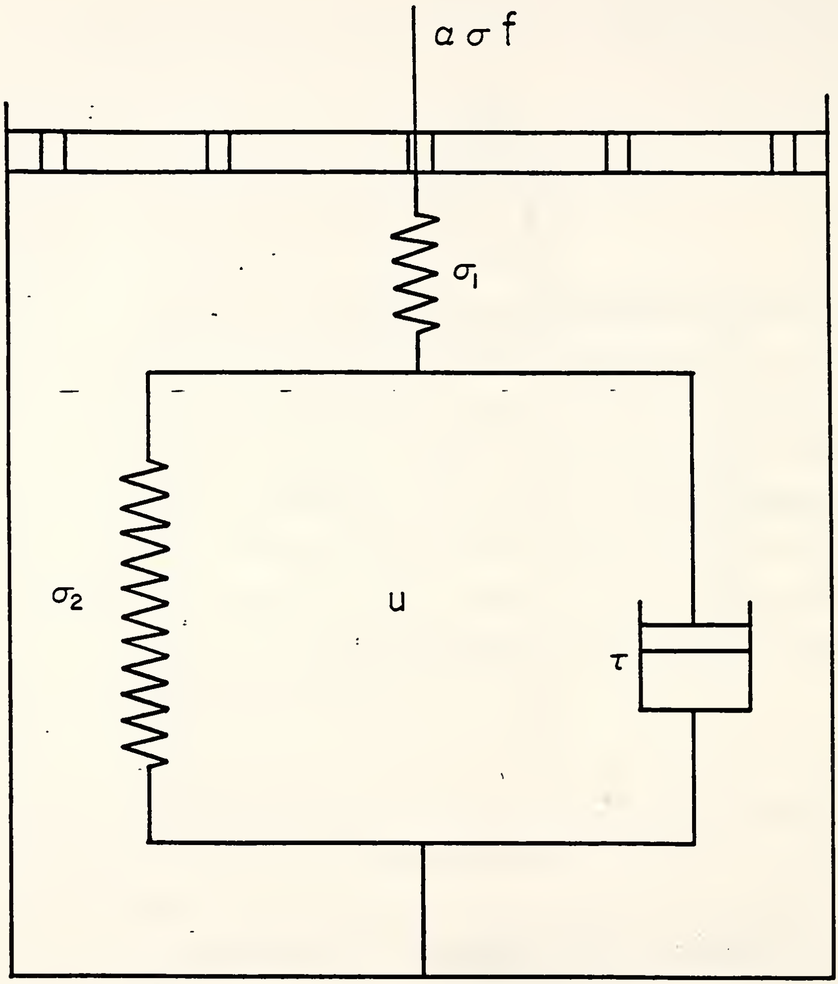


Figure 4.10 Berry and Poskitt's Model in a Terzaghi Pot.

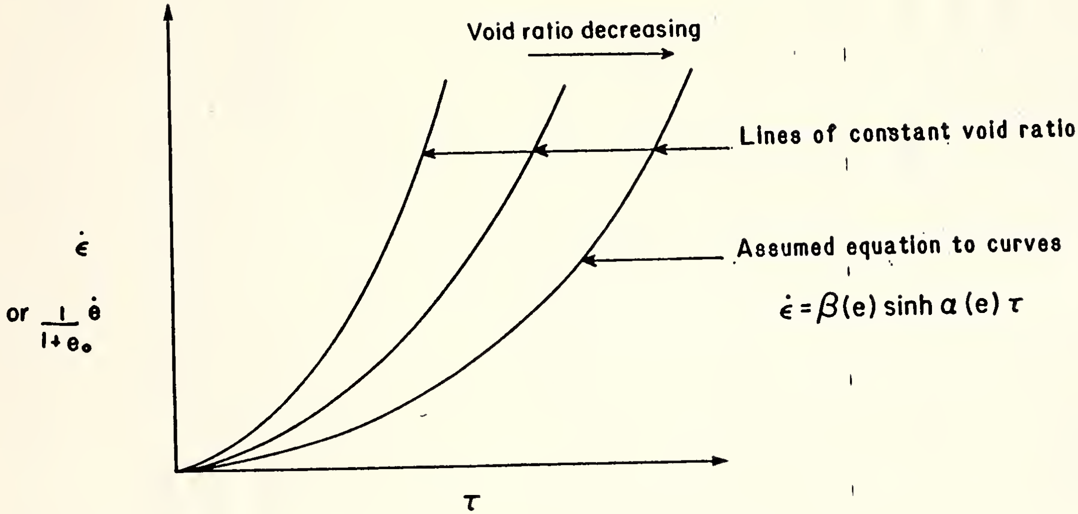


Figure 4.11 Relation between Stress and Strain Rate. From Berry and Poskitt (1972).

Based on the rate process theory, the secondary compression curves were described using equation 4.11:

$$\frac{1}{1+e_o}\frac{de}{dt} = -\beta(e)\sinh \alpha(e)\,\tau$$

where $\alpha(e)$ and $\beta(e)$ are rheological parameters which depend on the current value of the void ratio. Wu et al (1966) showed that this law predicted an essentially linear secondary compression relationship between the void ratio e and logarithm of time. For amorphous granular peat this has been found true for a fairly significant portion of the creep curve. Further, tests show that the relation holds also for the Indiana mucks tested.

Let the suffix 1 refer to the primary consolidation, the suffix 2 to the secondary compression part, the suffix 0 denote the initial condition, and the suffix f denote the final condition. Then,

$$e_o = e_{10} + e_{20} \tag{4.19}$$

$$e = e_1 + e_2 \tag{4.20}$$

If $\bar{\sigma}_1$ denotes the effective stress in the top spring and $\bar{\sigma}_2$ denotes the effective stress in the bottom spring, the stress in the dashpot is given by

$$\tau = \bar{\sigma}_1 - \bar{\sigma}_2 \tag{4.21}$$

Assuming a linear $e_2 - \log \bar{\sigma}$ relationship, equation 4.21 can be written as

$$\tau = \sigma_o \left[\left(\frac{\bar{\sigma}_f}{\bar{\sigma}_o}\right)^{\frac{e_{10}-e}{e_{10}-e_{1f}}} - \left(\frac{\bar{\sigma}_f}{\bar{\sigma}_o}\right)^{\frac{e_{20}-e_2}{e_{20}-e_{2f}}} \right] \tag{4.22}$$

Equation 4.11 must be written in terms of the secondary-compression-dependent component of the void ratio e_2. Hence, equation 4.11 becomes

$$\frac{1}{1+e_o} \frac{\partial e_2}{\partial t} = - \beta(e_2) \sinh \alpha(e_2) \tau \tag{4.23}$$

Consolidation tests by Wu et al (1966) show that $\beta(e)$ remains sensibly constant over a wide range of effective pressures and that $\alpha(e)$ decreases with increase in effective pressure. Hence, $\beta(e)$ has been assumed to be constant, and log α to follow a linear relation with e_2. Hence,

$$\alpha = \alpha_o \left(\frac{\alpha_f}{\alpha_o}\right)^{\frac{e_{20}-e_2}{e_{20}-e_{2f}}} \tag{4.24}$$

Substituting for τ and α in 4.23,

$$\frac{1}{1+e_o} \frac{\partial e_2}{\partial t} = - \beta(e_2) \sinh \alpha(e_2) \tau \tag{4.25}$$

Also, from Figure 4.8

$$u = \bar{\sigma}_f - \bar{\sigma}_1 \text{ and hence, } \frac{\partial u}{\partial z} = - \frac{\partial \bar{\sigma}_1}{\partial z} \tag{4.26}$$

Now, for a linear e_1 - log $\bar{\sigma}$ relation,

$$\bar{\sigma}_1 = \bar{\sigma}_o \left(\frac{\bar{\sigma}_f}{\bar{\sigma}_o}\right) \frac{e_{10}-e_1}{e_{10}-e_{1f}} \tag{4.27}$$

Taking logarithms on both sides

$$\log \bar{\sigma}_1 = \log \bar{\sigma}_o + \frac{e_{10}}{e_{10}-e_{1f}} \log \frac{\bar{\sigma}_f}{\bar{\sigma}_o} - \frac{e_1}{e_{10}-e_{1f}} \log \left(\frac{\bar{\sigma}_f}{\bar{\sigma}_o}\right)$$

Differentiating with respect to z,

$$\frac{1}{\sigma_1} \frac{\partial \bar{\sigma}_1}{\partial z} = - \frac{1}{e_{10}-e_{1f}} \log \left(\frac{\bar{\sigma}_f}{\bar{\sigma}_o}\right) \frac{\partial e_1}{\partial z}$$

$$\frac{\partial \bar{\sigma}_1}{\partial z} = - \frac{\bar{\sigma}_1}{e_{10}-e_f} \log \left(\frac{\bar{\sigma}_f}{\bar{\sigma}_o}\right) \frac{\partial e_1}{\partial z}$$

$$= - \frac{1}{e_{10}-e_{1f}} \bar{\sigma}_o \left(\frac{\bar{\sigma}_f}{\bar{\sigma}_o}\right)^{\frac{e_{10}-e_1}{e_{10}-e_{1f}}} \log \left(\frac{\sigma_f}{\bar{\sigma}_o}\right) \frac{\partial e_1}{\partial z}$$

$$= - \frac{1}{e_{10}-e_{1f}} \bar{\sigma}_o \log \frac{\bar{\sigma}_f}{\bar{\sigma}_o} \left(\frac{\bar{\sigma}_f}{\bar{\sigma}_o}\right)^{\frac{e_{10}-e_1}{e_{10}-e_{1f}}} \frac{\partial e_1}{\partial z}$$

$$\frac{\partial \bar{\sigma}_1}{\partial z} = - \frac{1}{e_{10}-e_{1f}} \bar{\sigma}_o \, 2.3 \log \frac{\bar{\sigma}_f}{\bar{\sigma}_o} \left(\frac{\bar{\sigma}_f}{\bar{\sigma}_o}\right)^{\frac{e_{10}-e_1}{e_{10}-e_{1f}}} \frac{\partial e_1}{\partial z} \tag{4.28}$$

Now

$$C_c = \frac{e_1-e_f}{\log \frac{\bar{\sigma}_f}{\bar{\sigma}_o}}$$ or,

$$\log \frac{\bar{\sigma}_f}{\bar{\sigma}_o} = \frac{e_o - e_f}{C_c}$$

$$\frac{\partial \bar{\sigma}_1}{\partial z} = - \frac{e_o - e_f}{e_{10} - e_{1f}} \frac{\bar{\sigma}_o \; 2.3}{C_c} \left(\frac{\bar{\sigma}_f}{\bar{\sigma}_o}\right)^{\frac{e_{10} - e_1}{e_{10} - e_{1f}}} \frac{\partial e_1}{\partial z}$$

$$\text{or} \quad \frac{\partial \bar{\sigma}_1}{\partial z} = - \frac{e_o - e_f}{e_{10} - e_{1f}} \frac{1}{a_{v_o}} \left(\frac{\bar{\sigma}_f}{\bar{\sigma}_o}\right)^{\frac{e_{10} - e_1}{e_{10} - e_{1f}}} \frac{\partial e_1}{\partial z} \qquad (4.29)$$

where $a_{v_o} = \dfrac{C_c}{2.3 \, \bar{\sigma}_o}$

Combining 4.26 and 4.29

$$\frac{\partial u}{\partial z} = \frac{e_o - e_f}{e_{10} - e_{1f}} \frac{1}{a_{v_o}} \left(\frac{\bar{\sigma}_f}{\bar{\sigma}_o}\right)^{\frac{e_{10} - e_1}{e_{10} - e_{1f}}} \frac{\partial e_1}{\partial z} \qquad (4.30)$$

where $a_{v_o} = \dfrac{C_c}{2.3 \, \bar{\sigma}_o}$

Substituting 4.20 and 4.30 into 4.18:

$$G \frac{\partial}{\partial z} \left[\frac{\left(\frac{k_f}{K_o}\right)^{\frac{e_o - (e_1 + e_2)}{e_o - e_f}} \cdot \left(\frac{\bar{\sigma}_f}{\bar{\sigma}_o}\right)^{\frac{e_{10} - e_1}{e_{10} - e_{1f}}}}{1 + e_1 + e_2} \frac{\partial e_1}{\partial z} \right] = \frac{\partial e_1}{\partial t} - \frac{\partial e_2}{\partial t} \qquad (4.31)$$

where $G = \dfrac{K_o}{a_{v_o} \, \gamma_w} \dfrac{(e_o - e_f)}{(e_{10} - e_{1f})}$

Equations 4.25 and 4.31 are the governing equations and being nonlinear, will have to be solved numerically.

THE MESRI-ROKHSAR THEORY

While not developed specifically for peats or mucks, the theory of consolidation as proposed by these authors was intended to be a generalized consolidation theory which took into account all observed soil behavior such as creep, decreasing permeability and varying compressibility. Again the basic equation used was the reduced form of equations 4.1 by Gibson et al, into which certain assumed constitutive equations were introduced.

The e-log σ curve was replaced by the bilinear relation shown in Fig. 4.1b.

Thus:

$$e_o - e = C_r \log \frac{\bar{\sigma}}{\bar{\sigma}_o}; \ \bar{\sigma} < \bar{\sigma}_c \tag{4.32a}$$

and

$$e_o - e = C_r \log \frac{\bar{\sigma}_c}{\bar{\sigma}_o} + C_c \log \frac{\bar{\sigma}}{\bar{\sigma}_c}; \ \bar{\sigma} > \bar{\sigma}_c \tag{4.32b}$$

where e is the void ratio corresponding to a stress $\bar{\sigma}$, C_r and C_c are the recompression and compression indices respectively and $\bar{\sigma}_c$ and $\bar{\sigma}_o$ are the critical and initial effective stresses, respectively.

A linear e-log k relation was also assumed and hence, equation 4.17 i.e.,

$$k = k_o \left(\frac{k_f}{k_o}\right)^{\frac{e_o - e}{e_o - e_f}}$$

remains valid. Substituting this equation into equation 4.16 written in terms of the Lagrangian (initial) coordinates and differentiating the resulting equation is:

$$D \frac{1+e_o}{1+e} b^{\beta} E \left[\left(\ln b + \frac{e_o - e_p}{1+e}\right) \frac{\partial \beta}{\partial \lambda} \frac{\partial \mu}{\partial \lambda} + \frac{\partial^2 \mu}{\partial \lambda^2}\right] = \left(\frac{\partial \beta}{\partial T}\right)_p \tag{4.33a}$$

where

$$D = \frac{0.434}{E} \left(1 - \frac{\bar{\sigma}_f}{\bar{\sigma}_o}\right) \tag{4.33b}$$

and μ, λ and T are dimensionless parameters defined as:

$$\mu = \frac{\bar{u}}{\Delta\sigma}$$

$$\lambda = \frac{z}{H_o}$$

and $$T = \frac{k_o\ (1+e_o)\ \bar{\sigma}_o}{0.434\ \gamma_w\ C_c} \frac{t}{H_o^2}$$

To account for creep effects, the soil compressibility was assumed to be a function of both effective stress and time. Hence,

$$\frac{\partial e}{\partial t} = \left(\frac{\partial e}{\partial t}\right)_p + \left(\frac{\partial e}{\partial t}\right)_{\bar{\sigma}}$$

$$\text{or } \frac{\partial e}{\partial t} = \left(\frac{\partial e}{\partial \bar{\sigma}}\right)_t \left(\frac{\partial \bar{\sigma}}{\partial t}\right) + \left(\frac{\partial e}{\partial t}\right)_{\bar{\sigma}} \qquad (4.34)$$

where $\left(\frac{\partial e}{\partial t}\right)_p$, $\left(\frac{\partial e}{\partial \bar{\sigma}}\right)_t$ and $\left(\frac{\partial e}{\partial t}\right)_{\bar{\sigma}}$ are the primary compression, the soil compressibility independent of time, and the secondary compression, respectively.

In essence, both primary consolidation and secondary effects begin simultaneously with the application of a pressure increment. The secondary compression was assumed to follow the relation

$$\Delta e = \frac{e_o - e}{e_o - e_p} C_\alpha \log t \qquad (4.35)$$

Equation 4.35 predicts a linear e-log t relation after primary consolidation is over, the rate being determined by C_α as defined in equation 4.12. During primary compression, the secondary compression is also dependent on the degree of compression $(e_o - e)/(e_o - e_p)$.

Differentiating equations 4.32, and 4.35 with respect to effective stress and time respectively

$$\left(\frac{\partial e}{\partial \bar{\sigma}}\right)_t = -\frac{0.434\ C_r}{\bar{\sigma}};\ \bar{\sigma} < \bar{\sigma}_c \qquad (4.36a)$$

$$\left(\frac{\partial e}{\partial \bar{\sigma}}\right)_t = -\frac{0.434\ C_c}{\bar{\sigma}};\ \bar{\sigma} > \bar{\sigma}_c \qquad (4.36b)$$

and

$$\left(\frac{\partial e}{\partial t}\right)_{\bar{\sigma}} = -\frac{0.434\ \beta\ C_\alpha}{t} \qquad (4.36c)$$

Further, excess pore pressure and effective stress were assumed to be related as

$$\frac{\partial \bar{u}}{\partial t} = -\frac{\partial \bar{\sigma}}{\partial t} \qquad (4.37)$$

Substituting equations 4.36 and 4.37 into 4.34, and expressing the results in terms of dimensionless parameters

$$\frac{\partial \mu}{\partial T} = \frac{1}{D\,\frac{C_r}{C_c}}\left[\mu + \frac{\bar{\sigma}_f}{\bar{\sigma}_o}(1-\mu)\right]\left(\frac{\partial \beta}{\partial T} - \frac{0.434\,\frac{C_\alpha}{C_c}\beta}{ET}\right)\ ;\ \bar{\sigma} < \bar{\sigma}c \qquad (4.38a)$$

and

$$\frac{\partial \mu}{\partial T} = \frac{1}{D}\left[\mu + \frac{\bar{\sigma}_f}{\bar{\sigma}_o}(1-\mu)\right]\left(\frac{\partial \beta}{\partial T} - \frac{0.434\,\frac{C_\alpha}{C_c}\beta}{ET}\right);\ \bar{\sigma} > \bar{\sigma}_c \qquad (4.38b)$$

Equations 4.33 and 4.38 are the governing equations for consolidation, defining the void ratio and the excess pore water pressure. Being nonlinear, these equations have to be solved numerically.

The reason for presenting both of these theories in detail is that while the Berry-Poskitt theory was developed specifically for peats, the theory of Mesri and Rokhsar is considered as being a general theory, applicable to any soil, including peats, and is the basis of a computer program called ILLICON. This program is one of the very few available to predict the behavior of embankments on soils exhibiting secondary time effects.

APPLICATIONS

Berry and Poskitt (1972) compared predictions with actual tests, and concluded that their theory gave good results. In order to merge the curves of the two equations, a simplified curve fitting approach was used. Mesri and Rokhsar (1974) compared the two theories and concluded that though the agreement between the curves is good, the physical significance of a slight bend in the initial portion of the pore pressure curves of Berry and Poskitt's (1972) theory is not obvious. The comparisons are shown in Fig. 4.12.

The Mesri and Rokhsar theory was developed into a computer program called ILLICON. Each soil layer within a stratum is divided into a number of sublayers, each with its own consolidation properties. At the boundary layers, the boundary

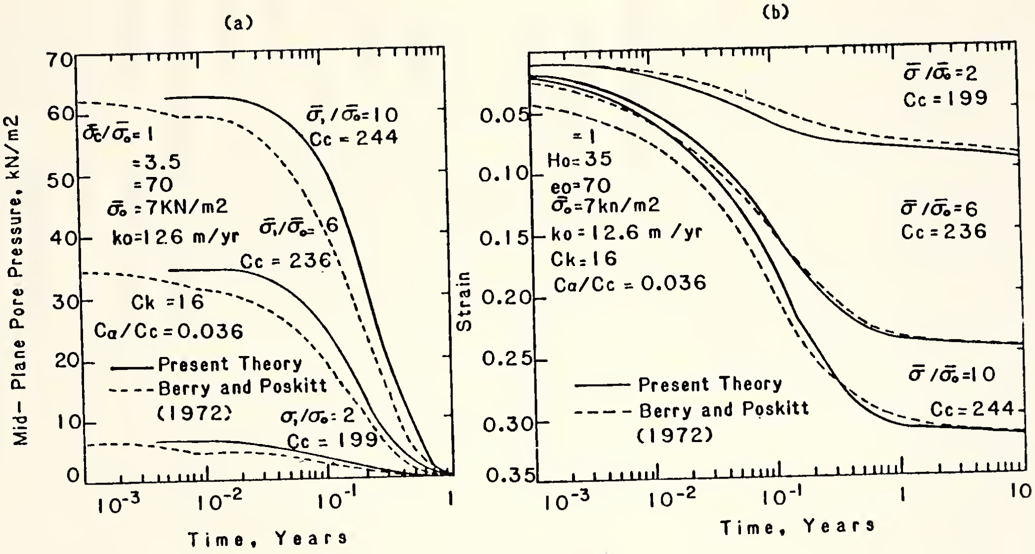


Figure 4.12 Comparison of Theory of Mesri and Rokhsar with Theory of Berry and Poskitt. From Mesri and Rokhsar (1974).

conditions of equal excess pore water pressure and equal flow rates must be satisfied. Any schedule of loading such as multi-stage embankment construction can be analyzed. The initial effective vertical stress is calculated from the total unit weight and sublayer thickness and the pre-construction in situ pore pressures. The final effective vertical stress is given by

$$\bar{\sigma}_{v_f} = \bar{\sigma}_{v_o} + \Delta\sigma_v$$

If the embankment is wide, $\Delta\sigma_v$ is equal to the pressure applied on the surface of the compressible soil layer. For loading over a finite area, $\Delta\sigma_v$ can be calculated using an elastic stress distribution. Also for wide embankments, $\Delta\bar{u}$ is taken equal to $\Delta\sigma$ though other assumptions about the excess pore water pressure such as $\Delta\bar{u} = \Delta\sigma_{oct}$ can be made.

Mesri and Choi (1985) used this program to analyze two embankments, one of which was the test fill at Vasby, whose anomalous behavior had been reported by Chang (1969), (1981). They concluded that ILLICON was a reliable method for settlement analyses of embankments on soft clays. Since the theory on which the program is based is equally applicable to amorphous peats and mucks, there is no reason why it cannot be used for embankments on such soils. It was unfortunate that for the Olga B test embankment, which Mesri and Choi (1985) analyzed, the peat was removed to prepare the embankment for instrumentation.

The ILLICON program is based on a theory of consolidation that appears to be superior to that of Berry and Poskitt (1972). However, there is no reason why theories of creep more powerful than that used by Berry and Poskitt cannot be used in combination with the Gibson equation. Further, Gibson, Schiffman and Cargill (1981) and Cargill (1984) have accounted for self weight of the soil, but have no provision for secondary compression effects. The logical direction in which to proceed is to combine these approaches which account for self weight, with a suitable model to account for secondary compression.

COMMENTS ON THE PRACTICALITY

The one-dimensional theories presented above have formed an important contribution to soil mechanics. Though very powerful theories, and quite capable of simulating behavior observed in the laboratory, they are none-the-less subject to the following practical limitations that will arise in any field situation. The theory presented above describes an initial boundary value problem of the heat conduction type. Its validity depends on the accuracy with which the initial conditions, the boundary conditions, and the material properties are known. In the laboratory, these factors are known, or can be determined accurately. In the field, however, the situation is not so clear cut, as a deposit normally consists of layers, and the boundary conditions are not easy to define. Initial conditions too are difficult to determine. Other factors such as drainage path length and coefficient of consolidation are not easy to determine in advance, but are required as input.

Data to input for the material properties depend on the results of tests run in the laboratory. However, only a limited number of such tests can be run, and these provide only localized soil properties. In the case of peats and mucks, the situation is made worse because these materials exhibit a high degree of heterogeneity, even within a small deposit, in both the vertical and lateral direction. Hence, if representative properties of a deposit are required, a large number of accurate tests need to be run. In view of their low shear strength and high compressibility, peats and mucks are, as previously explained, extremely difficult to sample without much disturbance. Also, accurate testing is difficult, and as described, special apparatus and techniques are required.

In this context is becomes clear that for normal projects involving construction on such soils, use of an extremely sophisticated and complex approach is unjustified. Such analyses should be reserved for test fills on large projects, where extensive and sophisticated sampling and testing techniques can be used, and serve, to extend the state of the art.

An observational approach, on the other hand, offers significant advantages in that, except for the initial design, no assumptions about homogeneity, etc., need be made. It is taken for granted that the initial design is only an approximate one and is used merely as a guideline, which will be modified as the observational procedure comes into play.

A simplified nonlinear settlement calculation procedure for initial design purposes, based on work by Mesri and Choi (1985), will now be described. This procedure, when combined with any one of the observational methods to be described later, will form a complete design methodology for construction on peats and mucks.

A SIMPLIFIED NONLINEAR APPROACH

A number of consolidation tests run at Purdue University show that the e–log $\bar{\sigma}$ curve for peats and organic soils is highly nonlinear, especially beyond the critical pressure. In most cases the ratio of $\bar{\sigma}_c$ to $\bar{\sigma}_{v_o}$ is less than three, and therefore assumption of a linear e–log $\bar{\sigma}$ relation for the recompression range is justifiable. Hence, referring to Fig. 4.13, the actual e–log σ curve during the recompression range is replaced by a straight line starting at the point e_o, $\bar{\sigma}_{v_o}$ and with slope C_r. The point P is defined at the preconsolidation pressure $\bar{\sigma}_p$.

To account for the significant nonlinearity of the e–log $\bar{\sigma}$ curve beyond the critical pressure, the following approximation is made. A modified compression index C'_c, is defined as the slope of the line connecting the point P to the point on the e–log $\bar{\sigma}$ curve corresponding to the stress level of interest, for

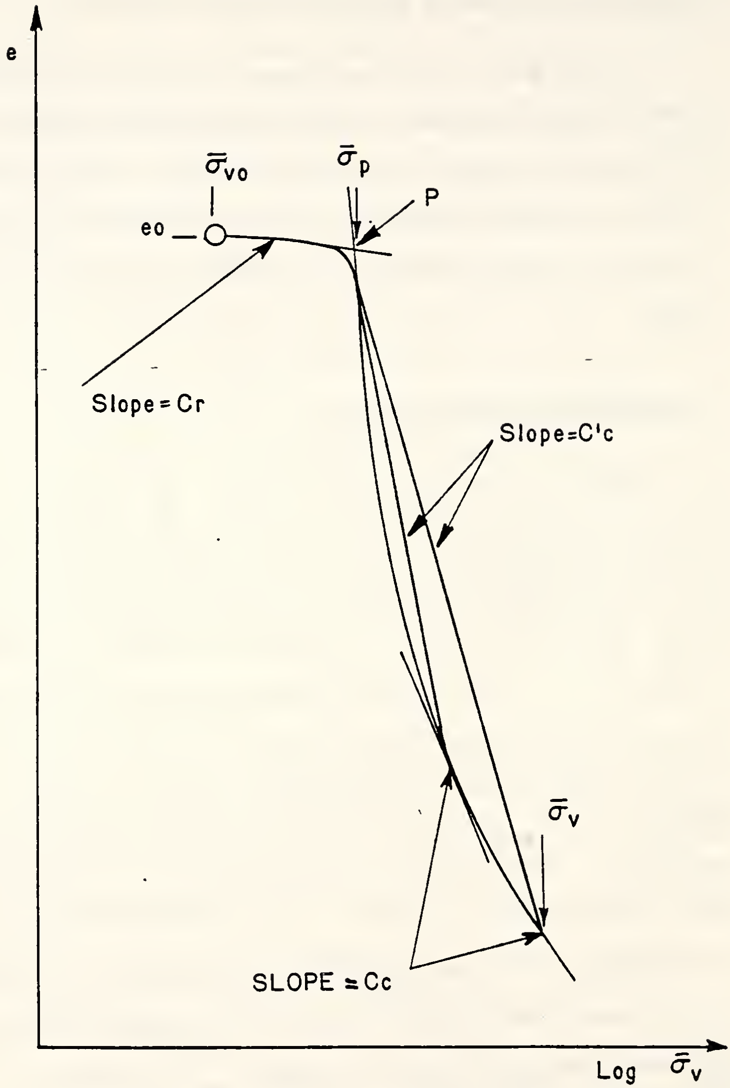


Figure 4.13 Definition of C_r, C_c and C'_c. From Mesri and Choi (1985).

example, $\bar{\sigma}_v$ in Fig. 4.13. For any particular consolidation test, C'_c can be determined at various levels of $\bar{\sigma}_v$ and, plotted against the corresponding $\bar{\sigma}_v$ values normalized with respect to the preconsolidation or critical stress $\bar{\sigma}_p$. This is shown in Fig. 4.14, where C'_c is plotted against log $\bar{\sigma}_v/\bar{\sigma}_p$, for various initial values of the void ratio. Thus, if any layer in a deposit is subdivided into sublayers of fairly constant void ratio, then for each layer, a curve of C'_c vs. log $\bar{\sigma}_v/\bar{\sigma}_p$ can be obtained. The resulting curves are shown in Fig. 4.14 where each curve corresponds to a particular sublayer and is obtained from the end-of-primary e-log $\bar{\sigma}$ curve obtained from a sample of that layer. In other words, the nonlinearity of the compressibility profile below the embankment is characterized by a set of C'_c-log $\bar{\sigma}_v/\bar{\sigma}_p$ curves versus e_o, as shown in Fig. 4.14. As three tests were run per layer, the resulting e-log $\bar{\sigma}$ curves were averaged in terms of C'_c and are shown in Fig. 5.5.

For any particular sublayer the values of e_o, $\bar{\sigma}_p$ and the final pressure $\bar{\sigma}_{vf}$ are known. The value of C'_c is read from the curve in Fig. 4.14 corresponding to the void ratio at this particular layer and to the appropriate log $\bar{\sigma}_v/\bar{\sigma}_p$ value. Using these values, the primary settlement(s) is calculated as:

$$s = \frac{L}{1+e_o} \left| C_r \log \frac{\bar{\sigma}_p}{\bar{\sigma}_{vo}} + C'_c \log \frac{\bar{\sigma}_{vf}}{\bar{\sigma}_p} \right| \qquad (4.39)$$

where L is the thickness of the sublayer. The process is repeated for all the layers and the results summed up to give the

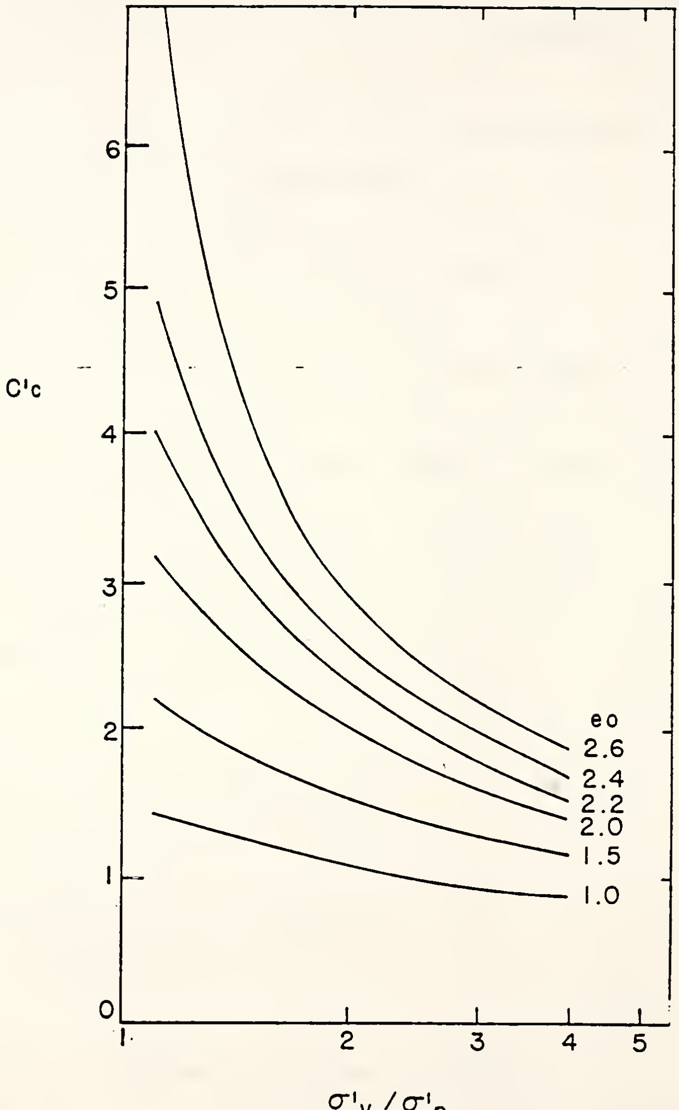


Figure 4.14 C'_c versus log σ'_v/σ'_p. From Mesri and Choi (1985).

value of the total primary settlement to be expected, under conditions of one-dimensionality.

Mesri and Godlewski (1977) concluded that C_α/C_c is a constant at all stresses, void ratios, and times, as previously discussed. For a variety of natural soils including peats, organic silts, highly sensitive clays, as well as granular materials, the value of C_α/C_c lies between 0.02 to 0.10, with the higher values being characteristic of peats and mucks. For a majority of inorganic soft clays,

$$\frac{C_\alpha}{C_c} = 0.04 \pm 0.01$$

and for highly organic plastic clays

$$\frac{C_\alpha}{C_c} = 0.05 \pm 0.01$$

For peats and mucks the value is higher and has a larger scatter (Chapter 5 and Appendix A). The value of C_α/C_c for a soil can be established from incremental loading tests, with C_α being determined by allowing secondary compression at three or four selected consolidation pressures. The values of C_c, corresponding to the same consolidation pressures, are determined from the end-of-primary e-log $\bar{\sigma}$ relations. A plot of C_α versus C_c should be used to determine C_α/C_c, as shown in Fig. 4.6. Once this value is known, the procedure described earlier can be used to draw the e-log $\bar{\sigma}$ curves at 10 t_p, 100 t_p, etc., and hence, the variation in C_α with time can be obtained. For most practical purposes, C_α can be assumed constant. The exception is when $\bar{\sigma}_{vf}$ is just beyond $\bar{\sigma}_p$. In such a case, t_p is small, as it occurs mostly in the overconsolidated range, and C_α is large and increases with time.

The secondary settlement for any layer with a constant C_α is given by

$$s_{sec} = \frac{C_\alpha}{1+e_p} (L - \bar{s}) \log \frac{t_f}{t_p} \qquad (4.40)$$

where t_f is the time at which s_{sec} is to be found, e_p is the void ratio at the end of primary compression, and is given by

$$e_p = e_o - C_r \log \frac{\bar{\sigma}_p}{\bar{\sigma}_{vo}} - C'_c \log \frac{\bar{\sigma}_v}{\bar{\sigma}_p} \qquad (4.41)$$

L is the original drainage length, and s is the primary settlement as defined by equation 4.39.

If C_α varies with time, the time interval between t_f and t_p is subdivided into intervals over which C_α remains constant, and the settlement for any sublayer is calculated as

$$s_{sec_{i+1}} = \frac{C_{\alpha_{i+1}}}{1+e_i} (L - s_i) \log \frac{t_{i+1}}{t_i} \qquad (4.42)$$

where $s_{sec_{i+1}}$ = secondary settlement at the end of the $i+1^{th}$ time interval,

e_i = void ratio at the end of the i^{th} time increment $= e_{i-1} - C_{\alpha_i} \log \frac{t_i}{t_{i-1}}$

L = initial length of drainage path

s_i = total settlement (primary and secondary) at the end of the i^{th} increment

t_{i+1} = time at the end of the $i+1^{th}$ increment and

t_i = time at the end of the i^{th} increment

- - - - - - - - - -

Using this approach, the total secondary settlement for each layer can be calculated using equation 4.40 if C_α is constant with time, or using equation 4.42 if C_α varies with time. The secondary settlements over all the layers are then summed to calculated the total secondary settlement. This is added to the primary settlement, to obtain the value of the total settlement after time t_f. A simple method of approximately determining t_p (the time required for primary consolidation in the field) will be presented.

PREDICTION OF FIELD TIME FOR CONSOLIDATION

Equations 4.38 can be solved numerically to obtain the time for consolidation in the field. An alternate approach has been to predict the time required for consolidation in the field, based on the time required in the laboratory, for the same load increment. The theory used in the prediction is that of Terzaghi, and was suggested by Hanrahan (1951). According to this theory, the time required to reach any given degree of consolidation is

$$t = \frac{T H^2}{c_v} \qquad (4.43)$$

where T is the dimensionless time factor, H is the length of the drainage path and c_v is the coefficient of vertical consolidation.

Hence,

$$\frac{T}{c_v} = \frac{t}{H^2} = \frac{t_1}{H_1^2} = \frac{t_f}{H_f^2} \qquad (4.44)$$

where the suffix l and f identify laboratory and field situations, respectively. Based on equation 4.44, the time required for primary consolidation in the field is evaluated as

$$t_{field} = t_{lab} \left(\frac{H_{field}}{H_{lab}}\right)^n \qquad (4.45)$$

where n = 2, and t_{lab} is the time required for primary consolidation in the laboratory, at the same stress level expected in the field.

However, based on field measurements the value of n for peats has been found to be less than two. Lake (1966) observed a value of n of 1, Lea and Brawner (1963) observed a value of 1.5, Hanrahan (1954), (1981) observed a value of 2.0, Samson and Rochelle (1972) observed values of 1.6 and 2.0, and Lefebvre et al (1984) observed values of 1.1 and 1.5.

Berry and Poskitt (1972) state that much discussion has arisen as to whether the primary consolidation time should be scaled according to $n = 2$ or to a more general form where $1 < n < 2$.

Reasons for this discrepancy were attributed by Lefebvre et al (1984) to factors such as nonlinearity of the material characteristics, creep, changing length of the drainage path, and variation in the direction of drainage. By making use of nonlinear finite strain consolidation theory, taking into account the creep occurring during consolidation, it can be shown that equation 4.45 with $n = 2$ is a special case of the more general expression. It shows that several conditions have to be met in order that $n = 2$. Also, when creep is significant (as is the case for materials like peat), the value of n must be less than 2.

The theory of consolidation used is after Gibson et al (1967), with the assumption that self weight is negligible. However, unlike Gibson's theory, the material is assumed to have intrinsic time effects, of the form proposed by Mesri and Rokhsar (1974).

If self weight is neglected, the governing equation becomes equation 4.16, which when written in terms of the void ratio becomes

$$\frac{\partial}{\partial z}\left[\frac{k(e)}{\gamma_w(1+e)}\frac{d\bar{\sigma}}{de}\right]\frac{\partial e}{\partial z} + \frac{\partial e}{\partial t} = 0$$

or

$$\frac{\partial}{\partial z} [g(e)] \frac{\partial e}{\partial z} + \frac{\partial e}{\partial t} = 0 \qquad (4.46a)$$

where

$$g(e) = \frac{k(e)}{\gamma_w (1+e)} \frac{d\bar{\sigma}}{de} \qquad (4.46b)$$

and $0 < z < 1$ for $t > 0$.

Hence, $T = \frac{g(e)\,t}{h^2}$, where T is the dimensionless time factor, g(e) is as defined in equation 4.46b, t is actual time and h is the drainage length in Lagrangian coordinates. Now, if g(e) varies in the same manner in the field as it does in the laboratory,

$$t_{field} = \frac{g(e)_{field}}{g(e)_{lab}} \left(\frac{h_{field}}{h_{lab}}\right)^2 t_{lab} \qquad (4.47)$$

If strains are small, and the soil is free from secondary time effects, equation 4.47 would be reasonable. Equation 4.47 reduces to 4.45 if $g(e)_f$ equals $g(e)_1$. For this to be true, at the very minimum, both drainage and compression in the field should be in the vertical direction only. Equation 4.47 is valid if strains are small and the soil free from creep. But for peats and mucks, strains are not small and creep does occur. Therefore, both must be taken into account. Thus, the thickness of a layer at any instant during compression is the thickness at that instant if consolidation were the only process, less the reduction in thickness that occurred as a result of creep up to that time. The drainage path length at any time t during consolidation is $(H - \Delta H)$, where H is the soil layer thickness if only consolidation were occurring and ΔH is the reduction that occurred up to time t as a result of creep.

Mesri and Rokhsar (1974) showed that secondary compression during consolidation can be written as (Equation 4.35)

$$\Delta e_{creep} = \beta \, C_\alpha \log t$$

Hence,

$$\Delta H = H \, \beta \, \frac{C_\alpha}{1+e_o} \log t \qquad (4.48)$$

C_α is assumed to remain constant during the period of primary consolidation and $\beta = \frac{e_o - e}{e_o - e_p}$, where e_o and e_p are the initial and final void ratios. As β varies with time from 0 to 1 during consolidation, an average value of 1/2 will not depart from the true value very significantly. If the suffixes i and f denote initial and final conditions, respectively,

$$H_f = H_i - H_i \, \varepsilon_f$$

or

$$H_f = H_i \, (1 - \varepsilon_f) \qquad (4.49)$$

where ε_f = the final strain when consolidation is over.

If strains are finite, the thickness of a layer will vary with time from H_i to H_f, and an assumption of an average value of $(H_i + H_f)/2$, though not correct, should not be far from the true value. It should be noted that such as assumption (and also the assumption of $\beta = 0.5$), implies a linear variation of void ratio and thickness with time, which is obviously not true. However, since only an equivalent value is required, the assumed values are not thought to significantly differ from the true ones.

Using equation 4.49, the average thickness is given by

$$H = (H_i + H_i - H_i\ \varepsilon_f)/2$$

$$\text{or, } H = \frac{H_i}{2}(2 - \varepsilon_f) \qquad (4.50)$$

Combining equations 4.47, 4.48 and 4.50, we get the complete expression.

From this it can be seen that only if the material is free from secondary time effects, if $[g(e)]_{field}$ and $[g(e)]_{lab}$ are the same, and if for the same load increment the field strains and the laboratory strains are the same, the final equation will reduce to equation 4.45. Only under these circumstances can a value of $n = 2$ be expected.

Since peats and mucks have a highly nonlinear e-log p relation, exhibit large secondary time effects, and being very compressible undergo finite strains, it is very unlikely that the simplified equation (4.45) will hold. It is likely that the difference between the field valves and laboratory values of g(e) and ε_f will increase as the nonlinearity and compressibility increase. However, even if the values in the field and laboratory are the same, the presence of the creep term results in a reduction of t_{field}. Since in no case has the value of 'n' been found greater than two, it can be concluded that the influence of creep is strong enough to account for the various combinations of g(e) and the final strains ε_f.

If field and laboratory parameters and strains are assumed equal, the equation reduces to

$$t_{field} = t_{lab} \left[\frac{H_{i,field}}{H_{i,lab}}\right]^2 \left[\frac{1 - \frac{C_\alpha}{2(1+e_o} \log t_{field}}{1 - \frac{C_\alpha}{2(1+e_o} \log t_{lab}}\right]^2 \tag{4.51}$$

This equation can be solved by trial and error using a pocket calculator and using equation 4.45 to find an initial starting value. It should be noted that $C_{\alpha_{field}}$ for narrow embankments is usually larger than $C_{\alpha_{lab}}$ due to a departure in the field from the K_o conditions in the laboratory. In general, the higher the embankment, the higher are the shear stresses, and the greater the deviation between $C_{\alpha_{field}}$ and $C_{\alpha_{lab}}$.

CONCLUSION

Until the 1970's engineers thought that peats and mucks behaved in a fundamentally different way from other soils. However, the extended theories that accounted for creep, finite strains and nonlinearity showed that the behavior of these materials could be fitted into the general framework of classical soil mechanics. Indeed, the sophistication of present consolidation theories can be viewed as having resulted from efforts in trying to predict the behavior of such materials.

CHAPTER V - THE LABORATORY TESTING OF MUCKS AND AMORPHOUS PEATS

INTRODUCTION

The testing program was aimed at studying the behavior of organic materials and to see how their properties differed significantly from those of inorganic soils. In order to do this, materials whose organic contents were high enough for them to be classified as mucks and peats were tested. The tests run were a series of consolidation tests, a series of creep tests, a series of permeability tests and a series of $\overline{CK_oU}$ (consolidated under K_o conditions and sheared undrained) tests. All the tests, except for the shear strength tests were run on undisturbed samples.

The tests in general showed results that fit within conventional soil mechanics. Some factors found significant were the non-linearity of the e-log p relationship, the linearity of the secondary compression, the influence of the soil fabric of these materials, and also the fact that in spite of the inherently high variability of the soil, a unique strength line was easily definable in p′-q space. All tests were run on each of the soil types, with the exception of the shear strength tests. These tests were run only on one soil. The data were analyzed and compared with existing theories. The Gibson-Lo Model was fitted to the creep curves and also, the shear strength data were studied from the critical state soil mechanics view point.

THE MATERIALS STUDIED

A preliminary survey was made over northern Indiana, and various areas likely to contain materials of interest were identified, with the help of the Indiana Department of Highways. After locating these areas, samples were taken and brought to the laboratory for testing. The tests run on these samples were for the water content and the organic content. From these tests, three areas with soils whose organic contents were within the range of interest, and which contained no fibers were identified. These areas were a depression along Lindberg Road in Tippecanoe County, a part of the shore line of Otterbein Lake in Benton County and along the Strubhar Ditch in Cass County, all three counties being in northern Indiana.

The properties of the three materials are shown in Table 5.1. The sites were chosen so that a broad range of organic contents could be studied. Once the sites were selected, block samples were taken. Initially it was thought that since these materials were weak and situated in low lying areas with a high water table, the taking of block samples would not be easy. In

Table 5.1 Index Test Results.

QUANTITY	STRUBHAR DITCH	OTTERBEIN LAKE	LINDBERG ROAD
Water content (%)	125-135	365-465	130-140
Organic content (%)	32-45	55-80	20-35
Specific gravity	1.8-2.1	1.5-2.1	1.7-2.1
Liquid limit	-	-	-
Plastic limit	-	-	-
pH	6.75	6.75	6.5
Initial void ratio	2.8-3.2	10-21	2.9-4.7
Fiber content	Nil	Nil	Nil
Normally consolidated Strength	$c' = 0.0$ kPa	-	-
Parameters	$\phi' = 36.86^{o}$	-	-
ASTM Classification	clayey muck	peat	clayey muck

reality, however, though the material was soft, and highly compressible, it was able to support itself without any problem whatsoever. Water seeped in, especially for the sample at Otterbein Lake, but bailing out the water was sufficient to prevent any flooding. In all it was found that for fiberless materials, the higher the organic content, the easier it was to take block samples, as the material could be carved into a block of the desired size, without much effort. A good block sample could be obtained in a total time of two to three hours. The procedure and material used were as specified by Winterkorn and Fang (1975). The soil block was surrounded by the sides of a box with 0.5 inch between the soil and the box. The top 0.5 inch of the soil block was removed, and a micro crystalline wax (Standard Oil Company: Eskar 50) was poured on the sides and top. The lid was screwed on, the bottom of the soil column carefully sheared and the box inverted, after the wax had solidified. The same process was repeated on the other side, and the sample transported by a non commercial van to the laboratory for testing.

INDEX TESTS

The results of the index tests are shown in Table 5.1. Though the tests were on samples taken from a block measuring a foot in length, breadth and height, the data showed a high variability. This confirms the fact that so far as the index properties are concerned, these materials are highly variable.

The procedure used for determining the water content was the same as that used for inorganic soils, though some writers (Landva et al (1983b)) recommend a temperature of $90 \pm 5^{\circ}C$ instead of the usual $110^{\circ} \pm 5^{\circ}C$. The organic contents were obtained by heating the materials in a porcelain container at $550^{\circ}C$ in a muffle furnace. Prior to being used, the calibration curve of the furnace was checked. In order to minimize the effects of temperature gradients found in these furnaces, the samples were placed near the thermostat. Generally the time of five and a half hours and a temperature of $440^{\circ}C$ as recommended by Landva et al (1983b) was not sufficient for complete ashing.

A high scatter was obtained from the specific gravity tests. One quick method to find the specific gravity of these materials is the use of equation (Landva, (1980))

$$P_s = \frac{3.26}{2.18-m} \tag{5.1}$$

where m is the ash content. This equation was, however, found to give slightly lower values as compared with tests using kerosene or water. One quick way is to use a gas pyconometer. This, however, requires some specialized equipment.

CONSOLIDATION TESTING

A series of consolidation tests was run on these materials to determine the e-log p curve. In order to reduce the disturbance of the sample, the following procedure was used. A stainless steel ring of thickness 0.020 inch was machined to form a tight fit inside the oedometer wall. The ring is shown in Fig. 5.1. It was pressed into the block sample, scooped out, trimmed and placed in the oedometer ring. While this method can be criticized for several reasons, it was found to be a practical and economical alternative to using the method proposed by Landva, or to using disturbed samples. The samples were saturated under a back pressure of 70 psi for several days, but, owing to the low initial degree of saturation (40-50%), complete saturation was not possible, and the degree of saturation decreased with increase in void ratio.

Peats and mucks are often described in the literature as being normally consolidated, since the usual e-log p curve shows no sudden break. This, however, is because the first load increment usually exceeds the preconsolidation pressure. This preconsolidation pressure is actually the result of creep deformations, snow load, etc. In order to define this preconsolidation pressure, very small load increment ratios (0.1 or less) were used in the early stages of loading, and, for each soil a preconsolidation pressure of about 7 to 10 kPa was obtained, after which there was a sudden break in the e-log p curve. This is shown in Figures 5.2 to 5.4. These figures are the average of values obtained from three tests per material. This approach was used because it was difficult to determine a unique void ratio for each stress level. Since consolidation occurred very rapidly, the exact end of consolidation was not

Figure 5.1 Ring Used to Take Undisturbed Samples.

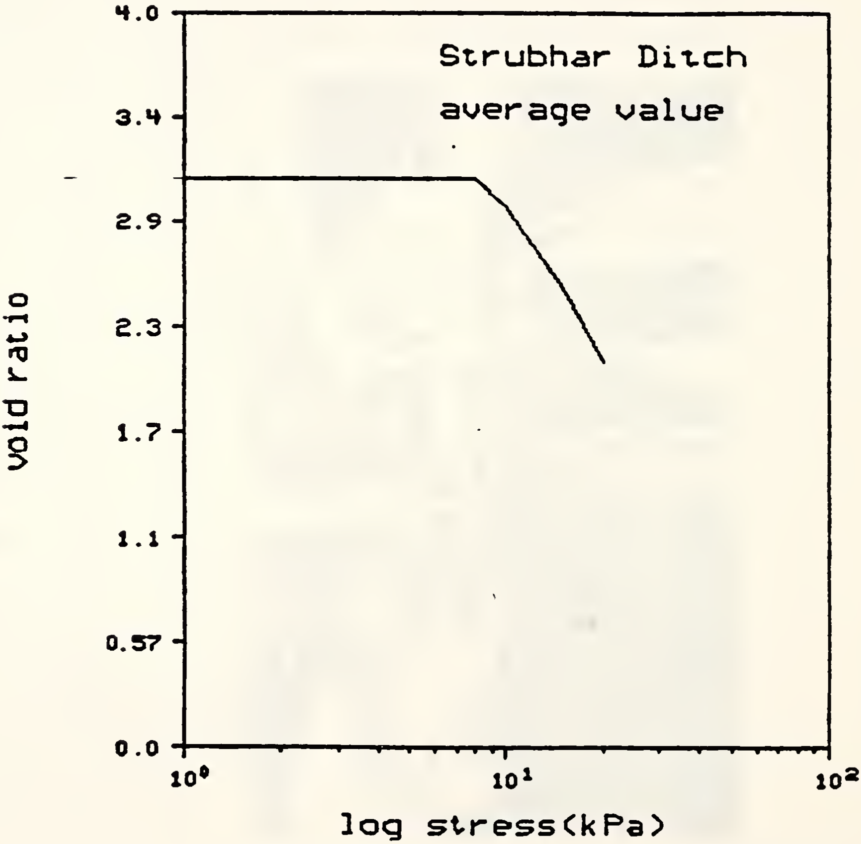


Figure 5.2 e-log $\bar{\sigma}$ Curve for Strubhar Ditch Material.

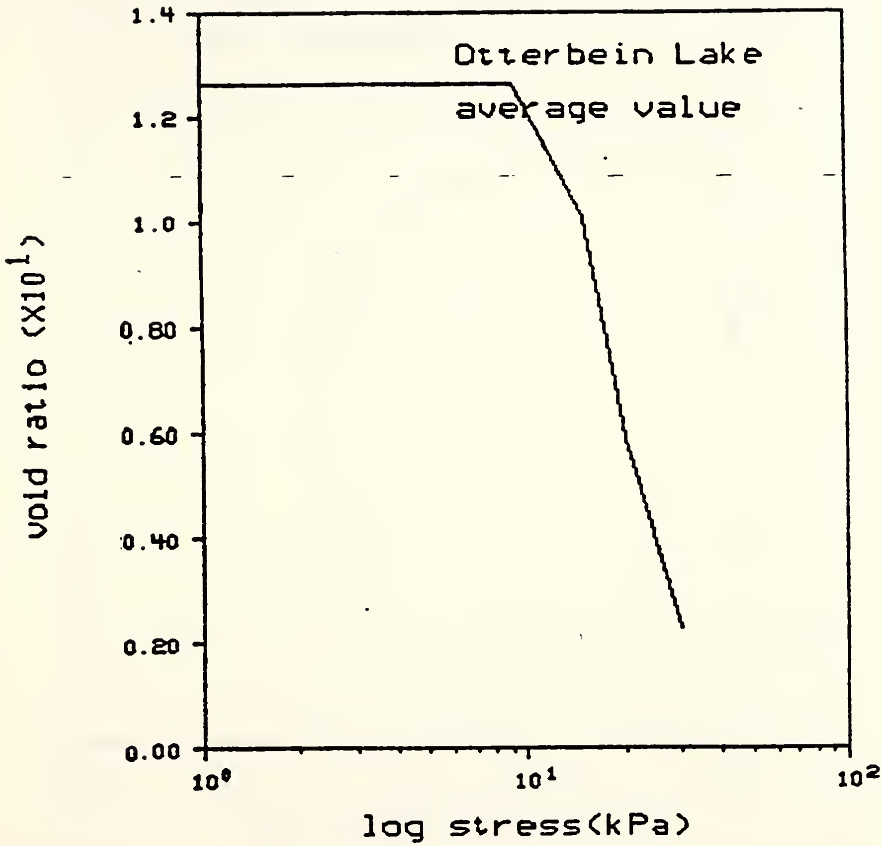


Figure 5.3 e-log $\bar{\sigma}$ Curve for Otterbein Lake Material.

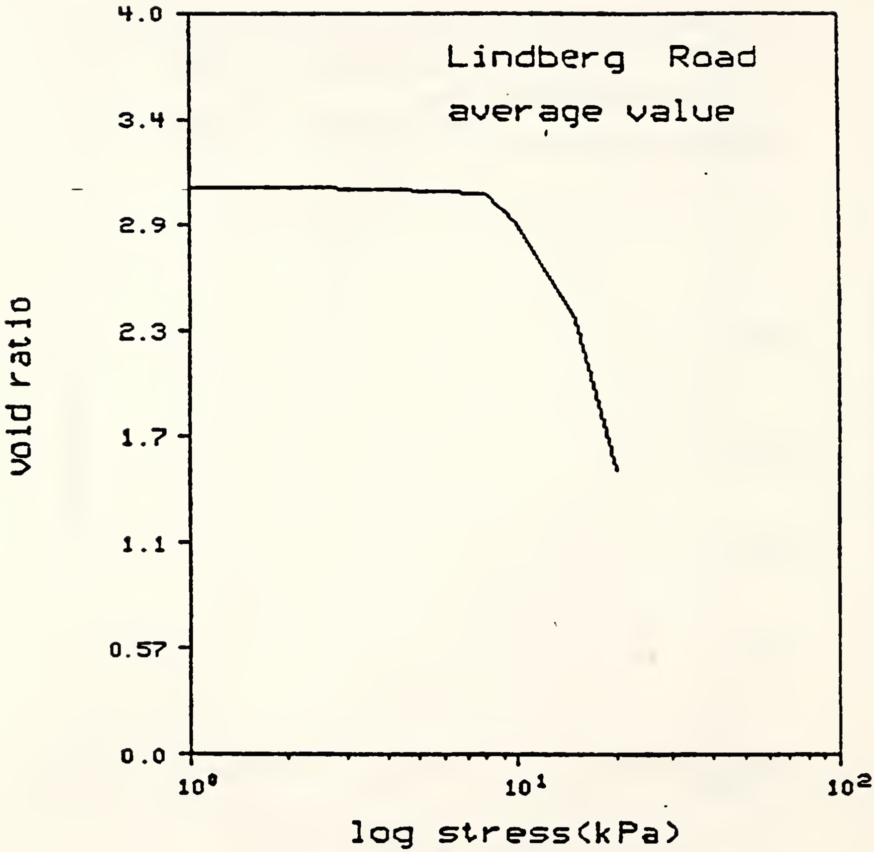


Figure 5.4 e-log $\bar{\sigma}$ Curve for Lindberg Road Material.

possible to determine. This is due to the large permeability and consequently the high c_v of the undisturbed material.

Weber (1969) suggested that for each load increment, the dial gage reading be taken at a fixed time after application of the load, and this value be used to define the e-log p curve. For the soils tested, consolidation was so rapid that no readings could be taken. A strip chart recorder was later tried, but was not of much help. All the drainage lines were closed prior to the application of each load increment, and in most cases pore water pressure rose by an amount equal to the applied stress. The time lag is probably due to the fact that saturation was not complete.

Fig. 5.2 to 5.4 show quite clearly that up to the preconsolidation pressure, very little deformation occurs. Beyond the preconsolidation pressure, however, compressibility is high and also not uniform. Of particular interest is test no. 3 on soil from the Strubhar Ditch, where the load increment beyond 8.5 kPa resulted in a large change in void ratio, probably as a result of destructuration. This existence of a definite structure is further confirmed from experiences with remoulded samples sedimented from slurry. For these materials, the void ratio at any given stress level is smaller than that of the natural undisturbed material, and with a permeability that is several orders of magnitude less. From this it can be indirectly concluded that while sampling disturbance is not considered important for fibrous peats, it may be of significant importance for amorphous materials.

Finally, the variation of C'_c with log σ_v/σ_p is shown in Fig. 5.5 where 1 is for material from Strubhar Ditch, 2 is for material from Lindberg Road and 3 is for material from Otterbein Lake. The variation of C'_c for these materials in general, and for the material from Otterbein Lake in particular show (a) the large variation in C'_c with σ_v/σ_p (as σ_p is small, relatively small changes in σ_v have an important effect), and (b) that the variation, unlike Fig. 4.14, is concave upward, which means that compressibility increases as void ratio decreases, which is not so. Probably, at higher stress levels, there is a sudden change in the e-log σ curve, and C'_c decreases rapidly.

CREEP TESTING

As previously mentioned, deformations under constant effective stress are of major importance for these materials. Hence, a series of creep tests were carried out, with each material being tested at three stress levels, and with a different sample for each stress level. The three stress levels chosen were: one below the preconsolidation pressure, one around the preconsolidation pressure, and one above the preconsolidation pressure. Since the preconsolidation pressure for all three materials was more or less the same, the three different stress levels chosen were the same for all three materials, viz., 2.27,

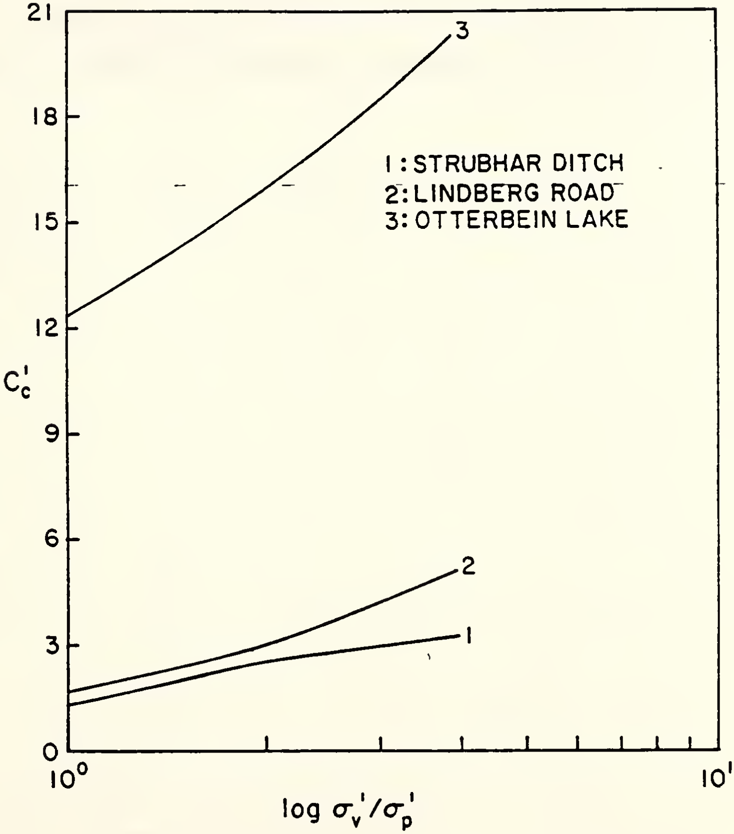


Figure 5.5 C'_c versus log σ'_v/σ'_p.

9.09 and 45.09 kPa. By choosing these stress levels it was hoped that various types of creep behavior including tertiary creep would be exhibited. The same procedure described earlier was used to install an undisturbed sample. Loads were applied using the normal lever arm type loading frame. In order to minimize the effects of temperature fluctuation, the frame was enclosed by insulated panels.

In general, it was found that irrespective of the stress level, the e-log t relation was almost perfectly linear. The test data are shown in the Appendix A and summarized in Table 5.2. The Gibson-Lo model was applied to the data, as shown in Fig. 5.6 to Fig. 5.11. In the figures, the 1 is for the curve using the values observed during the experiment, the 2 is for the curve using the Gibson-Lo model with parameters determined from the data of curve 1 (which was obtained at the same stress level) and 3 is for the e-log t curve predicted from the Gibson-Lo model using the parameters from a different stress level. It was found that the Gibson-Lo model was not suitable when strains are changing very slowly (below the preconsolidation pressure).

Further, the Gibson-Lo model gave good results only occasionally, ex. for the Otterbein Lake material, and then too only for parameters obtained for the same stress level. For the tests at 9.09 kPa on the Otterbein Lake and Strubhar Ditch material, it is shown that in the former case the predicted e-log t curve using parameters from the 45.49 stress level (curve 3) is

Table 5.2 Results from Creep Tests.

SOIL	STRESS (kPa)	e_o	e_f	DEGREE OF SATURATION (%)	C_α
Lindberg Road	2.27	4.70	4.69	51.99	0.03
Lindberg Road	9.09	3.60	3.59	68.59	0.06
Lindberg Road	45.49	4.60	2.89	55.16	0.10
Lindberg Road (repeat)	45.49	4.09	2.72	64.29	0.10
Strubhar Ditch	2.27	3.007	3.000	82.93	0.01
Strubhar Ditch	9.09	2.87	2.83	80.89	0.03
Strubhar Ditch	45.49	3.16	2.75	82.76	0.05
Otterbein Lake	2.27	13.73	13.70	51.68	0.03
Otterbein Lake	9.09	21.21	20.05	41.20	0.67
Otterbein Lake	45.49	10.91	10.09	61.05	0.15

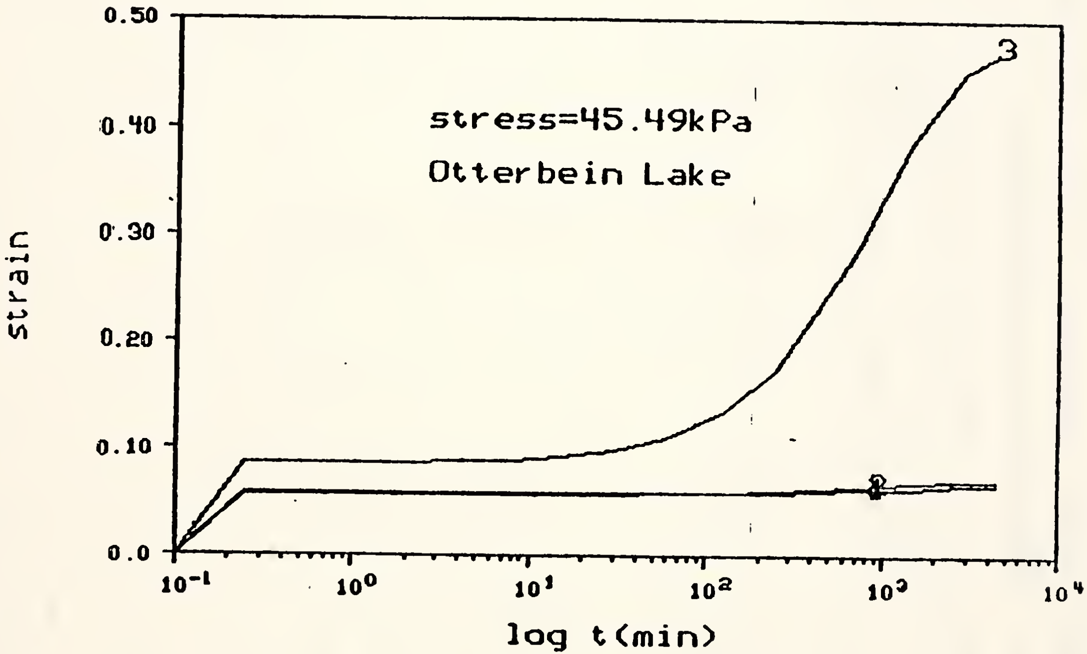


Figure 5-6 Gibson-Lo Model Fitted to Creep Data 03.

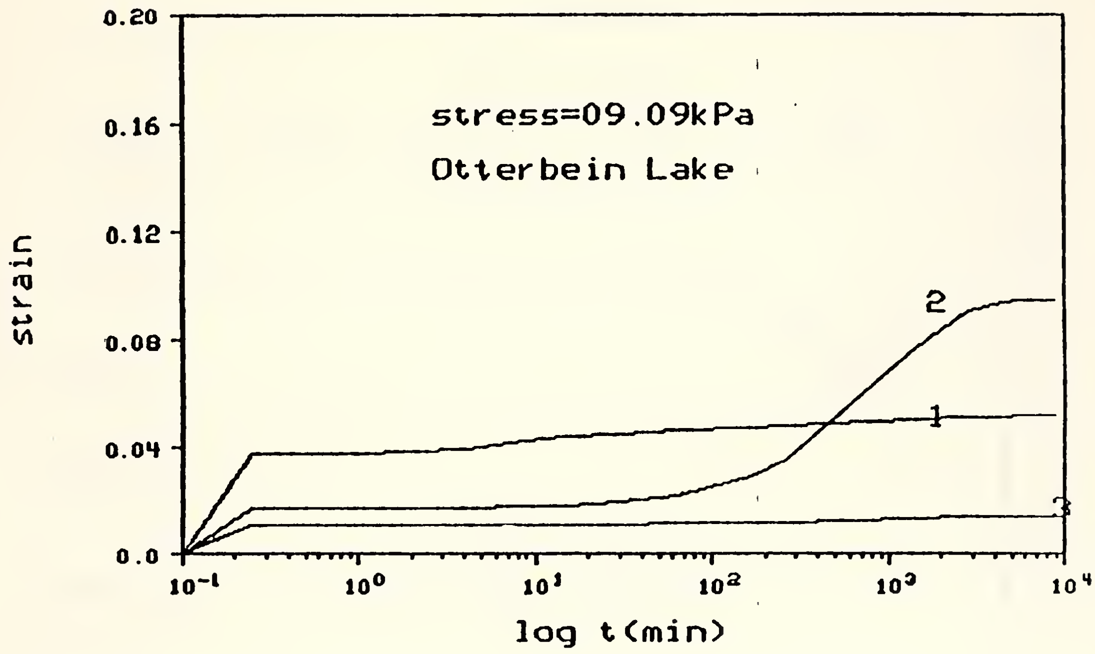


Figure 5.7 Gibson-Lo Model Fitted to Creep Data 02.

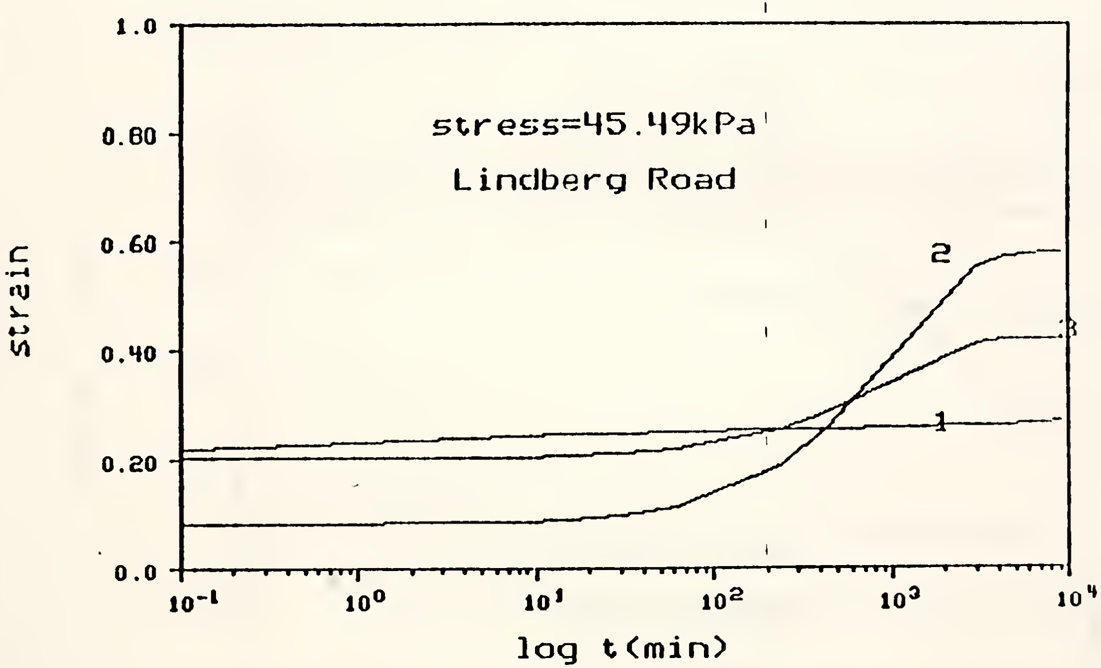


Figure 5.8 Gibson-Lo Model Fitted to Creep Data L3.

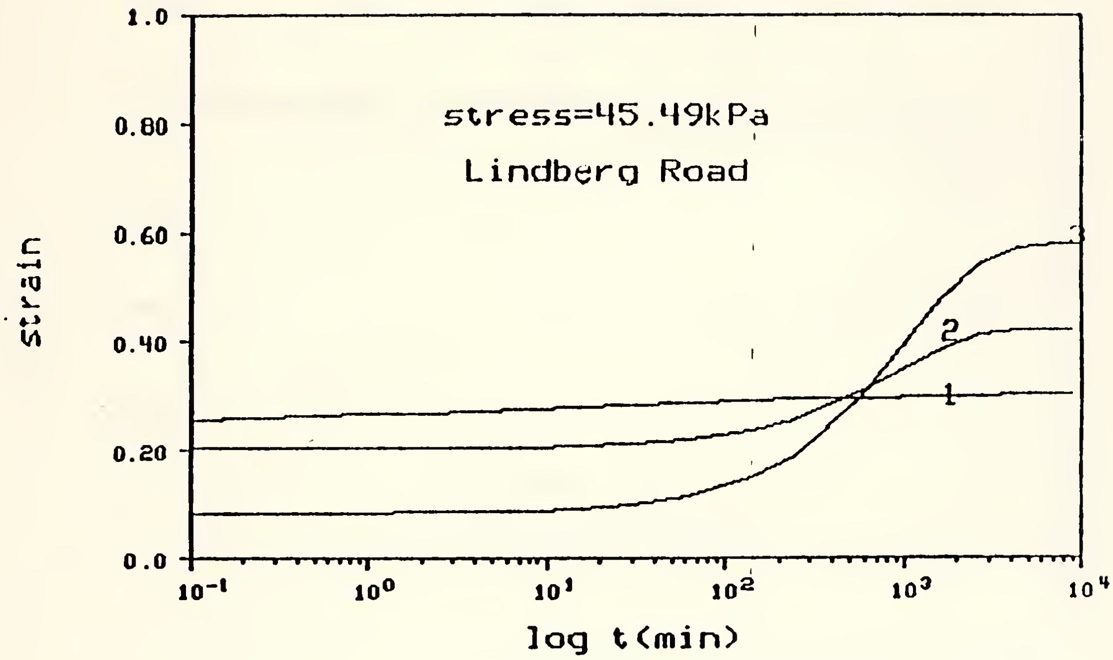


Figure 5.9 Gibson-Lo Model Fitted to Creep Data L3R.

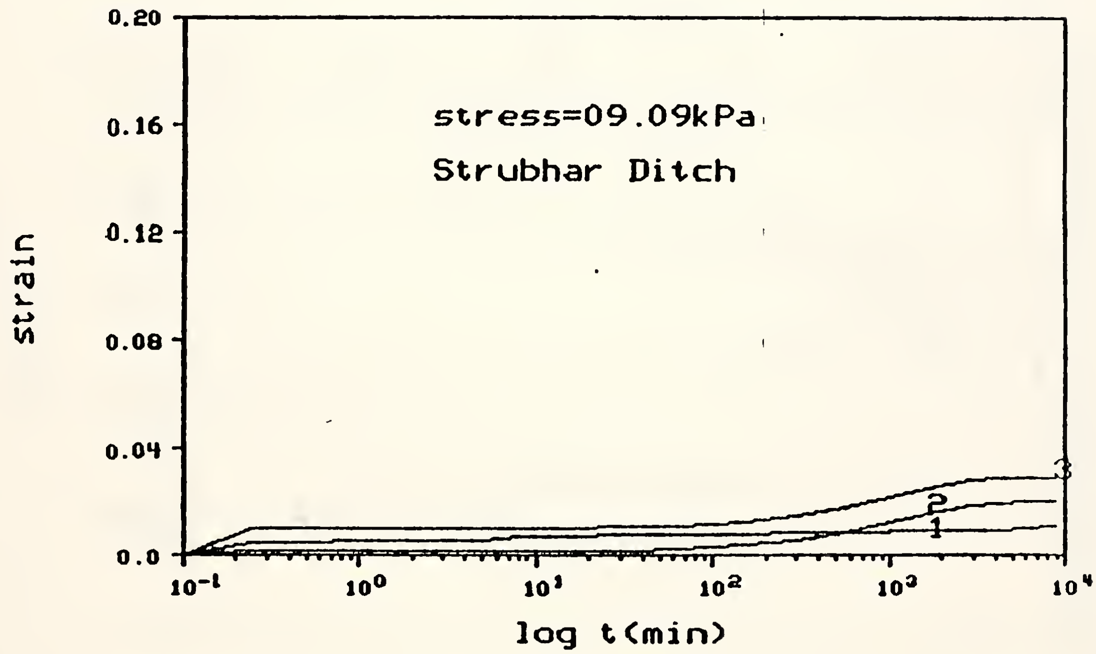


Figure 5.10 Gibson-Lo Model Fitted to Creep Data S2.

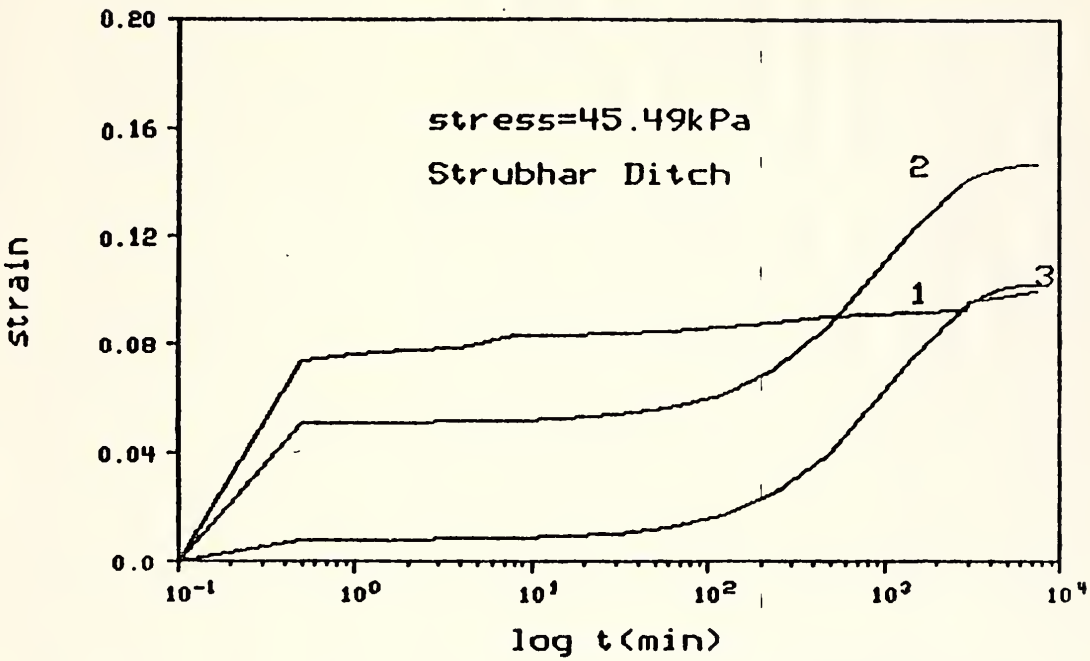


Figure 5.11 Gibson-Lo Model Fitted to Creep Data S3.

less than the actual strains at that stress (curve 1), while in the latter case the opposite is true. Thus, it can be said that the Gibson-Lo model is only an approximate one, and its chief attraction is that since the parameters involved are supposed to change very little with stress, it is ideal for the case of surcharge. It must be noted, however, that the parameters were obtained at a stress level five times greater or lesser (curves 2 and 3), and so the assumption of constant values for the Gibson-Lo parameters may not be valid. The parameters are shown in Table 5.3, and it is seen for Strubhar Ditch the parameters are relatively stress independent. Consequently for the lower stress level (9.09 kPa), where differences are not magnified, the model works quite well. In other words, the model will work reasonably well at low stress levels, using data from observations at slightly (1.5 to 2 times) higher stress levels.

When the stress level at which a prediction is to be made is high, and differs significantly from the stress level at which the parameters for the model were obtained, the prediction will not be valid. Hence, the Gibson-Lo model is suitable for embankments on peats and mucks, where final embankment heights are not large, and where the surcharge stress is generally about two times the final stress level.

Lo (1961) emphasizes the role of temperature fluctuations on creep tests. However, no such temperature dependence was observed for temperature fluctuations of up to $2^{\circ}C$.

Table 5.3 The Gibson-Lo Model's Parameters.

SOIL	STRESS (kPa)	a (m^2/kPa)	b (m^2/kPa)	λ (m^2/kPa)
STRUBHAR DITCH	9.09	1.81×10^{-3}	2.07×10^{-3}	1.75×10^{-6}
STRUBHAR DITCH	45.49	1.13×10^{-3}	2.10×10^{-3}	1.98×10^{-6}
OTTERBEIN LAKE	9.09	1.90×10^{-3}	8.47×10^{-3}	8.92×10^{-6}
OTTERBEIN LAKE	45.49	1.26×10^{-3}	3.07×10^{-4}	3.97×10^{-7}
LINDBERG ROAD	45.49	1.86×10^{-3}	1.08×10^{-2}	1.08×10^{-5}
LINDBERG ROAD (REPEAT)	45.49	4.49×10^{-3}	4.73×10^{-3}	5.41×10^{-6}

PERMEABILITY TESTING

These tests were run to determine the variation of the permeability with the void ratio. Permeability tests were run using the falling head test, with low head differences, in order to minimize the compressing effect of this test. Further, these tests were run on the samples at the end of the creep tests, when the change in void ratio was minimal. The permeability values obtained are the average of at least six tests run per sample at each stress level. The data are plotted in Fig. 5.12, and as can be seen they are more or less linear in e-log k space. Curves 1, 2 and 3 are for the materials from Lindberg Road, Strubhar Ditch and Otterbein Lake, respectively. The values of C_K are given in Table 5.4, where C_k is $\Delta e/\Delta \log k$.

SHEAR STRENGTH TESTING

As regards the shear strength of peat, most of the literature available, ex. Landva and La Rochelle (1983c), Edil and Dhowian (1980), etc. deals mostly with fibrous peats and not with the amorphous peats or mucks. In order to eliminate this gap, a series of undrained shear strength tests on samples consolidated under K_o conditions were run. As no data whatsoever were available, it was decided to run the tests on samples consolidated from a slurry. From rough initial calculations a quantity of soil was taken, mixed with water in a blender, to form a uniform mixture of the desired water content. The mixture

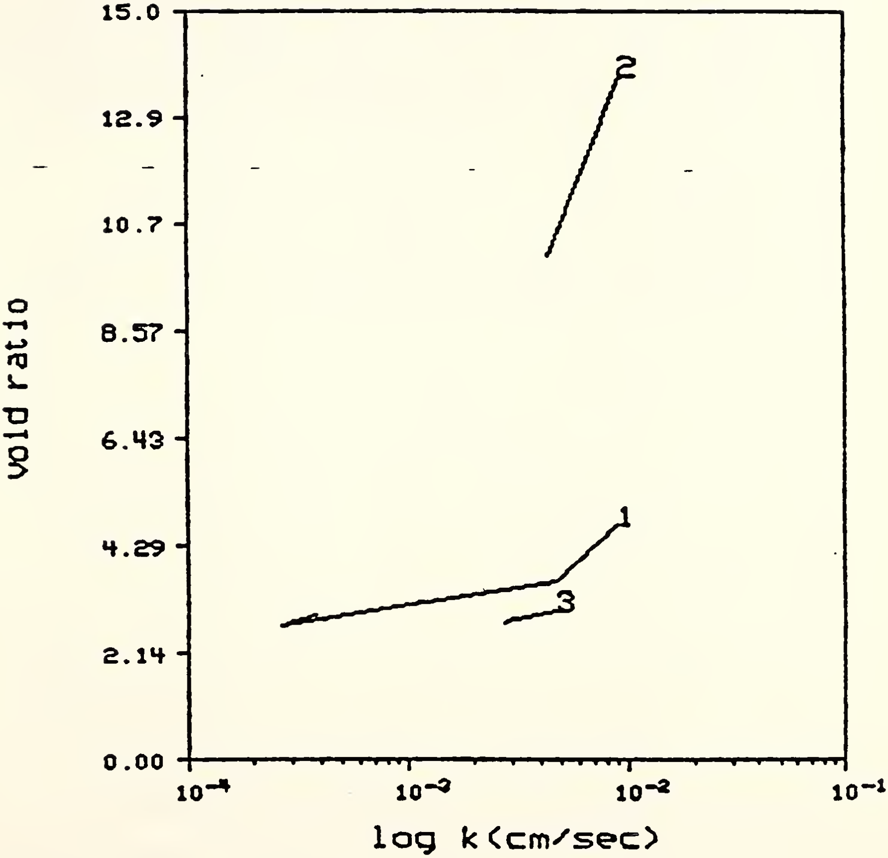


Figure 5.12 e-log k Relation for Soils Tested.

Table 5.4 Results of Permeability and Consolidation Tests.

SOIL	C_k	$\frac{C_\alpha}{C_c}$	$c_v(cm^2/sec)$
STRUBHAR DITCH	3.22	0.02	0.01
OTTERBEIN LAKE	9.64	0.04	0.01
LINDBERG ROAD	1.075	0.03	0.04

was placed in a beaker, and deaired under vacuum for not less than 24 hours, with occasional mixing. When deairing was complete (surface of slurry did not noticeably lower on removal of vacuum), the mixture was poured into a membrane held in the slurry consolidometer shown in Fig. 5.13, and consolidated at slightly less than the desired stress level. After consolidation was sufficient so that the sample had enough strength to be handled, it was mounted in the triaxial cell.

The triaxial cell is shown in Fig. 5.14, along with the control panel. The triaxial cell is designed after Campanella and Vaid (1972). The device allows consolidation to take place under K_o conditions by keeping the cell volume constant. This is done by keeping the piston and piston rod cross-sectional area equal to the sample area, so that increase of the cell volume by reduction of the sample height is compensated by an equal decrease in the cell volume. The decrease is caused by the intrusion of the piston as a result of the consolidation of the sample. Due compensation is made for intrusion of the bellow-frame into the cell space, by making the piston rod cross-sectional area slightly smaller than the sample area.

The sample height is approximately 6 inches and the diameter is 2.5 inches. The sample is installed, back pressure saturated at 100 psi for 12 hours, and consolidated to the desired stress level under K_o conditions (by locking cell drainage but allowing sample drainage). For the first twelve hours, drainage is

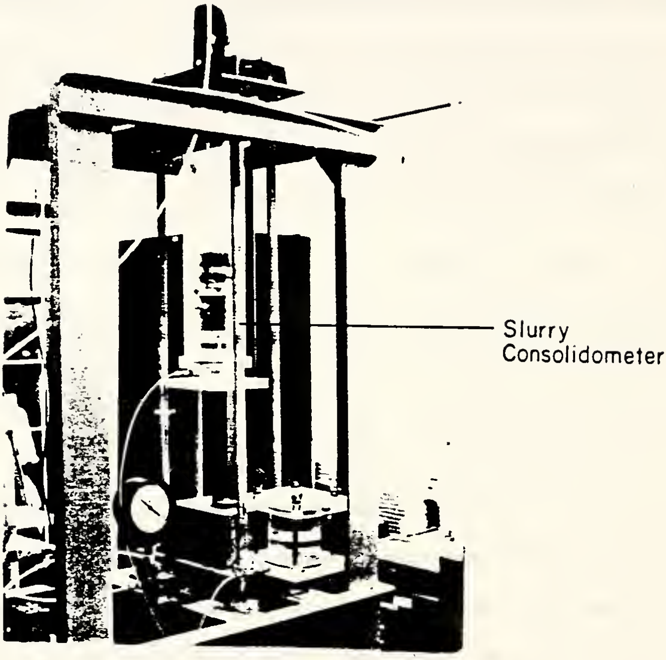


Figure 5.13 The Slurry Consolidometer.

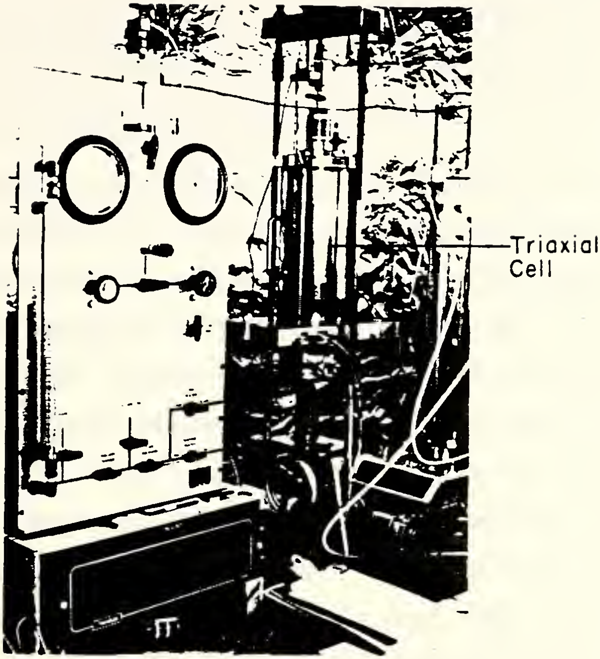


Figure 5.14 The K_o Triaxial Cell.

allowed only through the top, so as to make the void ratio uniform, after which drainage is allowed in both directions. The time required for consolidation was quite large, the reduced c_v values probably resulting from a lack of fabric. After consolidation was complete, the sample drainage lines were closed, and the sample was sheared (active compression).

In this program, six tests were carried out on the Strubhar Ditch material at various consolidation pressures and overconsolidation ratios. To check the behavior, one test was run on a normally consolidated sample and another on an overconsolidated sample of Lindberg Road material. The test data are shown in Appendix A and coded as follows: First comes the soil initial, i.e. S for Strubhar Ditch material, and L for Lindberg Road material. Then comes the test number for the particular soil. Hence, L2 stands for the second test on Lindberg soil. Next comes the overconsolidation ratio. This is followed by the consolidation pressure in psi, and by either ´CON´ meaning consolidation, or ´SHR´ meaning shear. Hence, S3-1-40.SHR stands for the third test on material from the Strubhar Ditch, tested at an OCR of 1, after consolidation to 40 psi, the data pertaining to the shearing of the sample.

It must be mentioned that all data are instantly converted from an analog signal to a digital signal, and stored in a computer. The testing equipment is very accurate, with very good repeatability. Each test took about 4 days to a week to complete.

As mentioned previously each sample was saturated at 100 psi for 12 hours. A `B' check at this stage gave B = 1.0 for all samples tested. After this, the cell pressure and the piston pressure were raised to the desired level and the sample was subjected to an isotropic stress state. The cell valve was closed, and the sample allowed to consolidate under K_o conditions. In all cases, both for loading and unloading, a classic type 1 curve was obtained when plotting e vs. log t. When consolidation was complete, the sample was sheared under undrained conditions at a strain rate of 0.0007 to 0.0008 inches/inch/sec. The results of the tests are shown in Appendix A, and the results are tabulated in Table 5.5. The results will be discussed in terms of the deviator stress-axial strain curve, the pore water pressure-axial strain curve and the p′-q stress path.

The deviator stress-strain curves are quite linear initially, with the undrained modulus, Eu, lying between approximately 3900 psi for S3-1-40 and 280 psi for S5-1-15. Failure was observed at very low strains, as is typical of K_o consolidated tests. For normally consolidated samples, peak strengths occurred at 1.5 to 2.7 percent strain. The strain at failure increased with overconsolidation ratio, and at an OCR of 10, was around 4.5 percent for the Strubhar Ditch material, and about 5.4 for the Lindberg Road material.

Table 5.5 Triaxial Test Data and Results.

TEST NO.	NAME	ε_{FAIL}	w_{ss}	e_{ss}	OCR	p'	q	ϕ'	K_{NC}	K_{OC}
1	S1-10-30	3.49	106.58	2.13	10	46.64	42.20	-	0.434	0.732
2	S2-1-30	1.56	104.4	2.09	1	139.8	90.22	37.69	0.549	0.549
3	L1-10-30	5.40	99.84	2.00	10	58.99	52.05	-	0.560	1.057
4	L2-1-30	2.13	92.57	1.85	1	145.8	105.5	-	0.488	0.488
5	S3-1-40	2.68	100.42	2.01	1	199.0	145.9	47.62	0.359	0.359
6	S4-2-30	3.55	108.96	2.18	2	117.6	88.26	30.96	0.551	0.789
7	S5-1-15	1.42	141.15	2.82	1	70.32	46.19	33.30	0.788	0.788
8	S6-10-30	4.58	141.76	2.84	10	65.83	59.31	30.96	0.626	1.043

The values of K_o for overconsolidated samples are generally less than or close to one. These low values have been observed by others, ex. Edil and Dhowian (1980). However, none of the usual models for determining $K_{o_{NC}}$ or $K_{o_{OC}}$ seemed applicable. The pore pressure response is shown in Appendix A. In each case, when the sample reaches its peak strength, there is a dip of around 2 kPa, indicating that at failure the sample tries to expand. It should be noticed that for normally consolidated samples, the pore pressure rises, and reaches a more or less constant value after failure. For overconsolidated soils the pore pressures were positive even at large strains and high overconsolidation ratios. This is true with the exception of test no. 1, which shows other anomalies, and being the first test run is considered as a trial.

Finally, the stress paths show some interesting results. When viewed from the critical state soil mechanics point of view, the stress paths of all the normally consolidated samples, for ex. S3-1-40 SHR, are more typical of lightly overconsolidated samples than the normally consolidated samples they undoubtedly were. This has also been observed in the field for inorganic clays. Becker et al (1984) state that their effective stress analysis of field loading conditions consistently showed lightly overconsolidated behavior, even though consolidation was not complete. For this test series, however, all samples were sheared only when excess pore pressures were around 2.0 kPa at the base of the 5 to 6 inch sample, or, in other words, consolidation was complete. Sample S2-1-30 SHR, however, shows relatively good correspondence, allowing for the fact that consolidation was under K_o and not isotropic conditions. Once the sample values reach the yield envelope, they display strain softening behavior, as shown in the stress paths for the normally consolidated samples, with the stress state tracking down the critical state line.

For overconsolidated samples, correspondence with the critical state model is quite good, especially for test S6-10-30 SHR. The stress path rises vertically, striking the Hvorslev surface, going up the Hvorslev surface (dilatent strain hardening behavior), and then striking the critical state line. This shows that as far as stress path behavior is concerned, the organic content seems to have no major effect.

A plot of the peak strengths in p'-q space is shown in Fig. 5.15, and reveals some interesting information. It shows very clearly that irrespective of its high variability as regards index properties, as far as peak shear strengths are concerned, they all plot on a unique line in p'-q space. This line passes through the origin, and while linear up to 30 psi, is thereafter, strongly concave upwards. The plot is shown in Fig. 5-15, with the test numbers at the coordinates representing the strengths for that test. This shows quite clearly, that the strength of amorphous organic materials is purely frictional. It further explains the high friction angles observed by other researchers

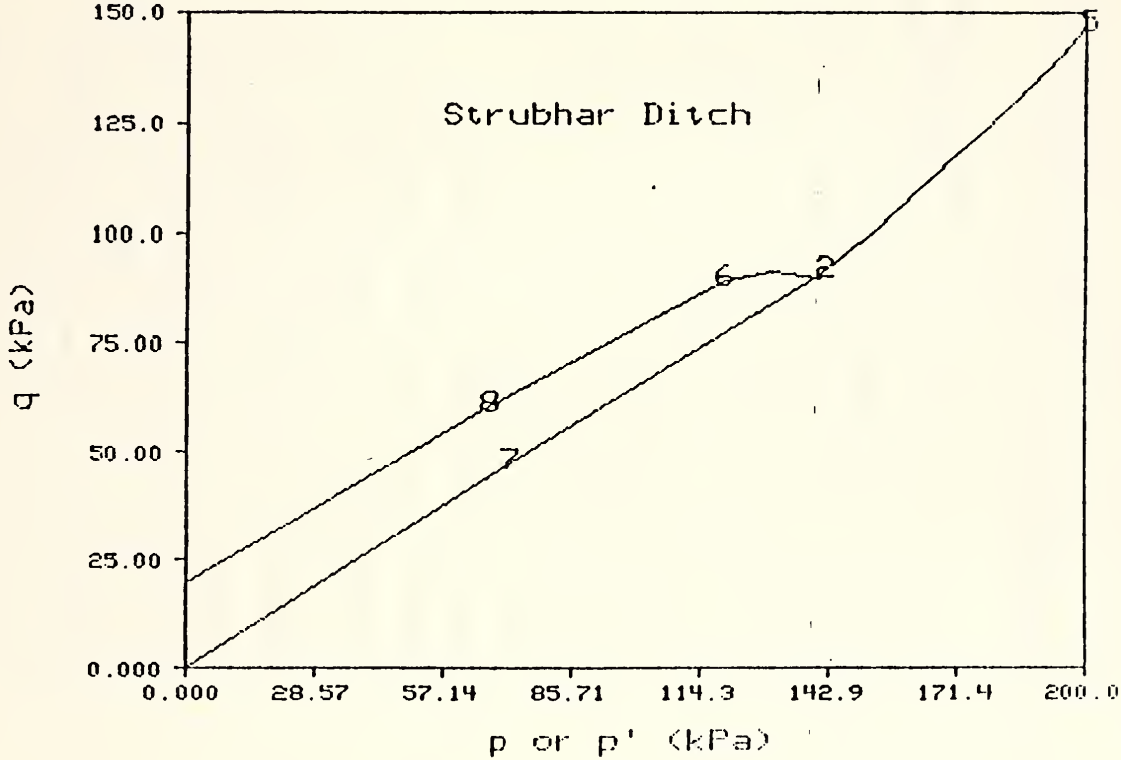


Figure 5.15 Failure Envelope in p'-q Space.

for ex. Edil and Dhowian (1980). The data and results from these tests are shown in Table 5.5.

CONCLUSIONS

The test program showed quite clearly that amorphous peats and mucks fit quite well within the general framework of soil mechanics. The e-log σ curve was, like that of a soft clay, nonlinear. Its nonlinearity could be expressed in terms of a C'_c vs. log σ_v/σ_p plot (Fig. 5.5) of the form proposed by Mesri and Choi (1985). The $\frac{C_\alpha}{C_c}$ value, while not easily definable for the recompression range, was more or less constant (Table 5.4). The creep tests showed behavior similar to many inorganic clays. The Gibson-Lo model was studied, and it was concluded that it was useful when the parameters were defined at a relatively close stress level, and where the stress levels were low. The permeability tests showed a fairly linear relation was obtained in e-log k space. Both the permeability tests and the e-log σ curves indicated indirectly that amorphous materials had a definite fabric. Finally, the shear strength tests showed that the critical state soil mechanics concept was valid at these high organic contents, that the failure envelope in p'-q space was unique, passed through the origin and was concave upwards, showing that the strength was purely frictional, and explaining the high friction values sometimes reported.

CHAPTER VI - THE BEHAVIOR OF EMBANKMENTS ON PEATS AND MUCKS

INTRODUCTION

Highway engineers, avoid routing highways through areas where the soils consist of peats of mucks, as far as possible. The reason for this is that such soils are weak, highly compressible and subject to considerable secondary compression, which often even exceeds the settlement due to primary consolidation. Various construction methods have been developed to deal with such soils. Of the various methods, the most frequently used method is to surcharge the soil.

This chapter describes the various construction methods used for building on such soils. The concept of the effective stress path and yield envelope is used to explain qualitatively the benefits of surcharging. The development of pore pressures during construction, and general behavior both during and after construction is studied. Various possible failure modes are examined. Various engineers from places all over the world have used different analytical methods to predict the behavior of embankments on such soils, with varying degrees of success. Their efforts and conclusions will be briefly presented. Also examined is the use of berms, drains, and lightweight fills.

CONSTRUCTION METHODS

There are several ways of building embankments on peats and mucks. All of these methods can be broadly classified as belonging to one of the following three approaches: (1) displacement, (2) floatation, and (3) surcharging.

The 'displacement approach' consists of displacing the peat or muck by good quality material. Casagrande (1966) summarized over three decades of experience with this method. He concluded that the most efficient way of constructing a displacement type embankment on such materials is to first destroy the surficial mat or crust of the organic soil deposit by blasting a 5 to 10 foot strip. The purpose of destroying the mat is to allow the fill to settle to the bottom. After this strip is destroyed, a fill is constructed by end-dumping over the blasted strip. This will cause the mat or crust to yield letting the fill sink to the bottom. This method is shown in Fig. 6.1, and was used extensively to build the German autobahns over such soils. It is no longer popular because it is costly, untidy, and not safe.

The reason this method is presented is two fold. First, some engineers recommend cutting of the mat before construction of the fill [Ex. Tressider (1958)]. MacFarlane and Rutka (1959) and Landva (1980) concluded that roads on a base mat performed better than those without a base mat. If the road is to be constructed by the displacement method then, as previously described, the mat will have to be cut. If the road is to be

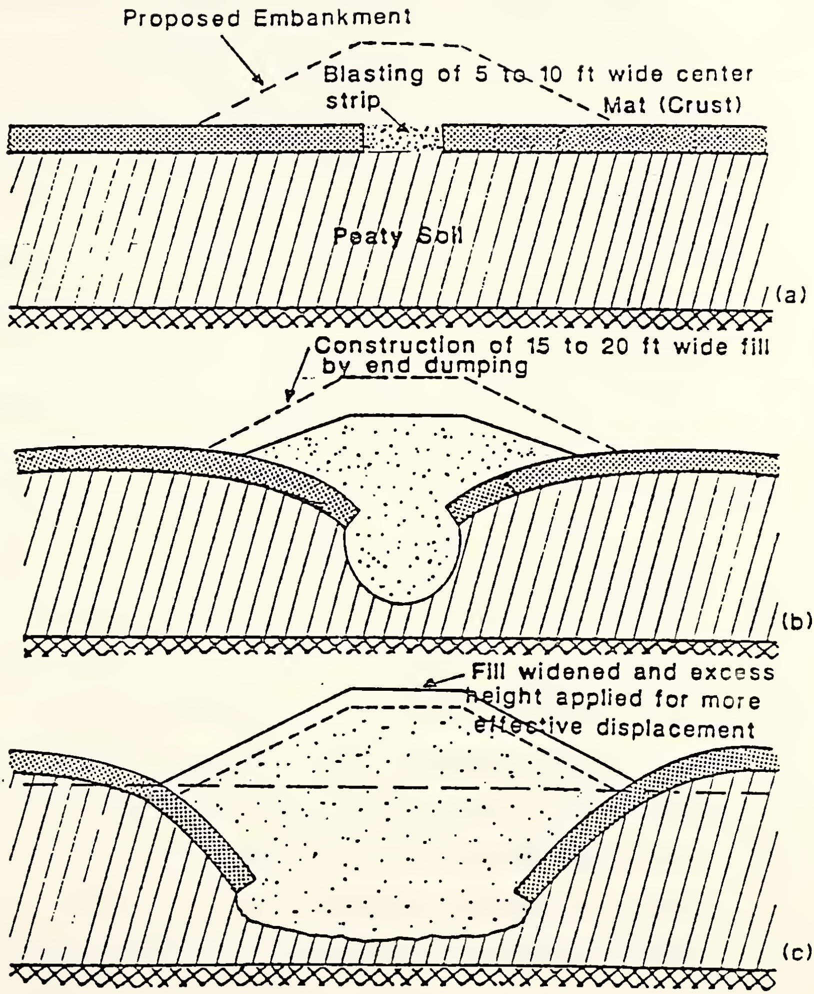


Figure 6.1 The Displacement Method. From Casagrande (1966).

´floated´ across the deposit, cutting the mat would be detrimental to performance. Secondly, many if not all of the predictive types of analysis are based on the assumption that plastic flow and lateral displacement of the type shown in Fig. 6.1 does not occur. If during surcharging, such a displacement occurs, there must be a discrepancy between predicted and observed values.

The ´floatation approach´ consists of building the embankment on a base layer which may be the surficial root mat, a geotextile, a layer of tree trunks or a combination of these. There are two misconceptions generally associated with this method. The first is that the road is buoyant on the surrounding soil. This is not so, because normally the unit weight of the embankment material is greater than that of the surrounding soil. The second misconception is that the bearing capacity is obtained from the tensile strength of the mat. Landva (1980) showed that such a strength contribution was negligible and that in the long run, the mat would slowly weaken and yield. Various analyses have shown [Rowe (1984a,b)] that to obtain any significant effect, the geotextile should be very stiff and be located as close to the surface as possible. The main effect of the textile was to reduce plastic failure and lateral deformation. The action of the geotextile is quite similar to that of the root mat. By preventing local bearing capacity failures (plastic failure) and lateral deformation (displacement), significant savings in the amount of fill used have resulted. Fowler (1985) reported the successful use of a geotextile on a muck, when other alternatives were not cost effective. However, long term data on the behavior of such materials are not yet available.

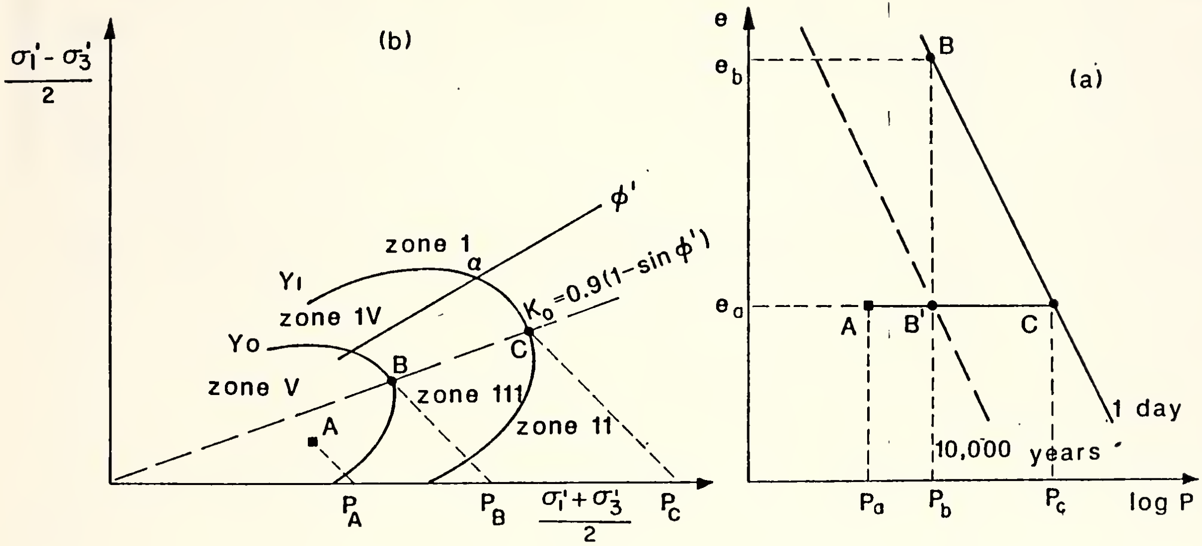
The third construction approach is to use a surcharge. This method has been used extensively for construction on soft clays and has been recommended for use on peats by several researchers such as Brawner (1959,1961), Weber (1969), Samson and La Rochelle (1972), Edil (1983), Gruen and Lovell (1983), Swantko et al (1984), etc. The reasons for the use of surcharging are to increase the soil strength, reduce compressibility and also the long term secondary compression. However, Brawner (1965) and Landva (1980) expressed doubts as to the benefits of surcharging. Landva stated that the main effect of the surcharge was to accelerate the process of consolidation. Since pore pressures in peat dissipate within 2-4 months after construction, he questioned the use of surcharge for peats.

In the next two sections the effective stress path/yield envelope concept is described. This concept provides a framework within which the process of stage construction and surcharging can be studied. Landva (1980) analyzed data presented by Samson and La Rochelle (1972) and concluded that the surcharge had no effect on the rate of secondary settlements. This too can be qualitatively explained using the yield envelope concept.

THE EFFECTIVE STRESS PATH AND YIELD ENVELOPE CONCEPT

Tavenas and Leroueil (1977) combined the principles of limit and critical state proposed by Roscoe and other workers, with the work of Bjerrum on the effects of aging and strain rate. On the basis of their experimental evidence, they concluded that their model could be used to qualitatively explain the behavior of natural Champlain clay. Tavenas and Leroueil (1980) generalized this model to include all soft clays. Since it was shown earlier that the consolidation of peat could be predicted by a generalized consolidation equation, there appears no reason why the yield envelope concept cannot be applied to amorphous peats and mucks.

Briefly, the model proposed by Tavenas and Leroueil (1977) is as follows. When a soil was deposited, its mineralogy and deposition environment determined the effective friction angle ϕ' and the e-log p relation. Continued deposition resulted in the clay being subject to increasing overburden stresses and accompanying horizontal stresses. As the stresses increased, the void ratio decreased along the e-log p line. In Fig. 6.2a, point B with void ratio e_B corresponds to the situation at the end of deposition. If a sample of any soil is subjected to a stress path, and if for this stress path the strain energy is plotted as a function of stress as shown in Fig. 6.3, a sudden break in the curve occurs. This break in the curve corresponds to a stress state at which yield occurred, with yield being defined as a



PROPOSED MODEL FOR THE BEHAVIOR OF CLAYS

Figure 6.2 Proposed Model for Clay Behavior. From Tavenas and Leroueil (1977).

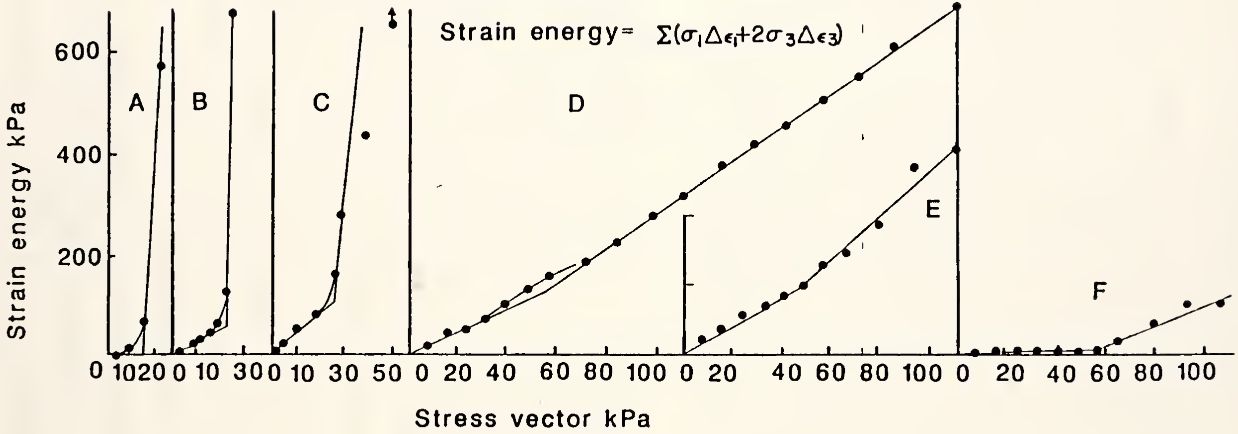


Figure 6.3 Yield Characteristics of Recent Sediments. From Watson et al (1984).

change from small strain response to large strain response to loading. If drained tests along various stress paths are conducted, it is possible to define a locus of stress states at which failure takes place. Such a locus is known as a yield locus.

Referring to Fig. 6.2b, let y_o be the yield locus corresponding to point B and the void ratio e_b. As shown in Fig. 6.2b, the yield locus is symmetric about the K_o line. Now as aging occurs, due to the process of secondary consolidation, the void ratio decreases over a period of 10,000 years. Let the void ratio decrease from e_b at point B to e_a at point B′ at the constant overburden pressure P_b. This would result in the development of a quasi-preconsolidation pressure P_c, corresponding to point C, along with a new yield locus corresponding to y_1. If some erosion occurred, a reduction of stress would result from P_b to say P_a, with point A in both Figures 6.2a and 6.2b representing the condition of the soil. Now, using Fig. 6.2b, the behavior of the soil can be explained.

For stress conditions within Zone 1, failure occurs immediately when the applied effective stresses correspond to points on the yield locus to the left of point α, or to points on the Mohr-Coulomb line to the right of point α.

For stress conditions corresponding to Zone 11, large consolidation settlements develop when the effective stress path crosses the yield envelope y_1 below point α. A test simulating this condition is the oedometer test for stress levels greater than P_c. As long as the stress level lies below the ϕ' line, failure will not occur.

For stress conditions between the y_o (young) yield locus and the y_1 (aged) yield locus, below the ϕ' line, secondary volumetric deformations occur at a rate which depends on the proximity of the applied stress state to y_1. Close to y_1, the deformation rate will be high. Close to y_o the deformation rate will be small, and will correspond to the existing rate of secondary compression. No failure can develop as stress conditions are below the ϕ' line.

For stress states below the yield locus y_1 and above the critical state line corresponding to Zone lV, the clay will initially remain stable. Gradually, however, creep deformations will develop and the yield locus will move from y_1 to y_o. As it moves to y_o it will pass through the applied stress state. The time required for the yield locus to move from y_1 through the effective stress state depends on the position of the stress state relative to y_1. When y_1 passes through the stress state, stresses correspond to a point on the yield locus to the left of the point α, and failure occurs. Since it occurred at a constant stress, it will appear to be a creep failure.

For stresses corresponding to Zone V, i.e. within the y_o yield locus, no failure will occur.

This model was generalized by Tavenas and Leroueil (1980) and used to analyze the behavior of embankments on clay. Watson et al (1984) used it for preloading and stage loading structures on soft soils in Trinidad. Crooks et al (1984) used it to analyze the pore pressure response of several embankments. Becker et al (1984) also used the generalized model to study strength gain due to consolidation.

PRELOADING AND STAGE LOADING

For soft soils, a commonly used construction procedure is staged loading and preloading. Sometimes, however, preloading has not resulted in any gain of strength with consolidation. Becker et al (1984) studied 19 case records of construction on a wide variety of soil types. In 10 cases strength increased on consolidation. In 6 cases, no strength gain was reported, while the data from the remaining 3 were inconclusive. They attributed the lack of strength gain to insufficient consolidation, as opposed to anomalous soil behavior.

Folkes and Crooks (1984) examined the behavior of several embankments and developed an effective stress path/yield envelope approach to qualitatively explain observed behavior. Their approach consists essentially of determining the effective stress paths at various points in the embankments foundation. The total stress at a piezometer tip can be determined. Knowing the pore pressures at this point, the effective stresses are known. The excess pore water pressure and yield behavior can be related to the location of the effective stress paths relative to the yield envelope. The approach, though qualitative, provides a rational framework for examining observed behavior. Thus far it has been used only where no principal stress rotation has occurred, i.e. only along the center line of the embankment.

The model used is a generalized version of that previously described [Tavenas and Leroueil (1977)] and is shown in Fig. 6.4. The yield envelope is the locus of stress states in p-q space at which yield response changes from one of small strain to one of large strain. This envelope is symmetric about the K_o line. Its apex cuts the K_o line at a point defined by the preconsolidation pressure. As previously mentioned it is normally obtained from drained triaxial tests, though the upper and lower portions of the envelope coincide approximately with the undrained shear strength in compression and extension. For effective stress paths within the yield envelope, and below the critical state line, the soil behaves in an overconsolidated manner with rapid dissipation of excess pore water and small strain response. If the stress state lies within the yield envelope, but above the critical state line, a meta stable condition exists. In this region, an increase in embankment height or an increase in pore water pressure will cause the effective stress path to fall to the critical state line and then move down the line. This strain softening stress path behavior is associated with large horizontal strains, pore water pressures and total horizontal

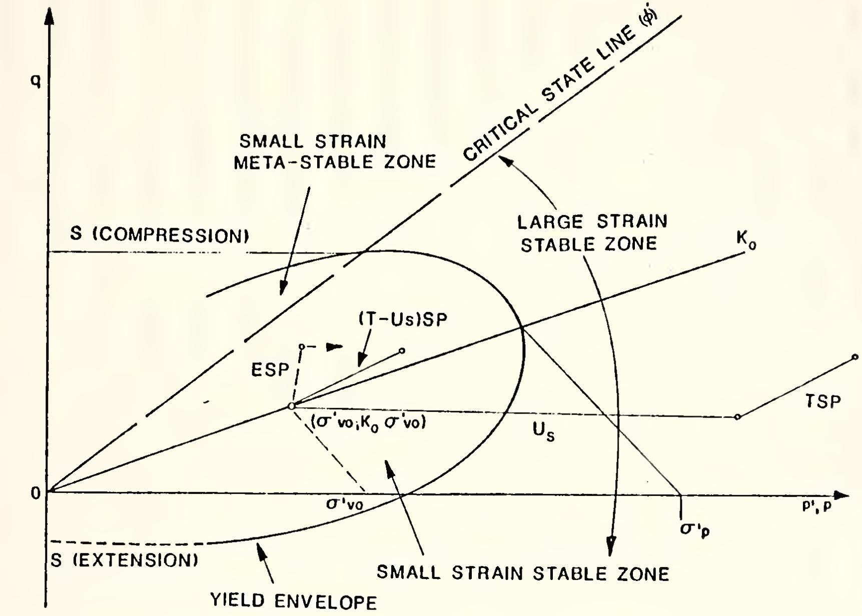


Figure 6.4 The Features of the Yield Envelope. From Crooks et al. (1984).

stresses. If the soil is not strain softening and the shear stress equals the shear strength, further loading will cause the pore pressures to increase so that the effective stress state remains constant. For effective stress paths outside the yield envelope and below the critical state line, behavior is normally consolidated. Loading is accompanied by large strains and pore water increases. The pore water dissipation rate is also reduced.

Becker et al (1984) used this model to examine strength gain due to consolidation. They concluded that where consolidation was sufficient to result in a new effective stress state which lay outside the original in situ yield envelope, strength gain would be evident. In other words, the final effective vertical stress should be greater than the preconsolidation pressure. If, at the time of testing, the effective stress state lies within the original yield envelope, no strength gain will be evident, as strength at this time is governed by the original preconsolidation pressure. If on the other hand, a large amount of strain softening has occurred, the old yield envelope will be destroyed and a new yield envelope, within the old envelope, and determined by the current state of effective stress, will control behavior. It is responsible for the lower post-construction strengths. Pore water pressure dissipation for strain softened materials takes place very slowly, i.e. the effective stress state can lie within the original yield envelope for a very long time. This is the reason why post construction strengths are sometimes lower than the initial strength. This condition will exist until the effective stress path crosses the original yield envelope.

Watson et al (1984) used the above concept to successfully preload and stage-load structures. Since strain softening results in considerable lateral deformations, they prevented strain softening by applying the stress path shown in Fig. 6.5. The first stage of loading is given by the total and effective paths OA and OA′. Further loading would have resulted in a path like OA′PR. Section PR represents strain softening and would have been accompanied by zones of localized yield and for further loading, large lateral deformations. On the other hand, by allowing some pore pressure dissipation to occur, the effective stress path moves from A′ to A″. At this point, the load can be increased to total and effective stresses represented by B and B′. By controlling the stage loading in this way, the effective stress path can be taken beyond the original yield envelope and below the Mohr-Coulomb line. For stress states within this zone, consolidation settlements will be large. Excessive lateral stresses will, however, be minimized as no significant strain softening is allowed to occur. This example describes the control exerted at one piezometer location. By controlling the stress paths in the same manner at a sufficient number of piezometer locations, the mass behavior of the subsoil beneath the foundation can be controlled.

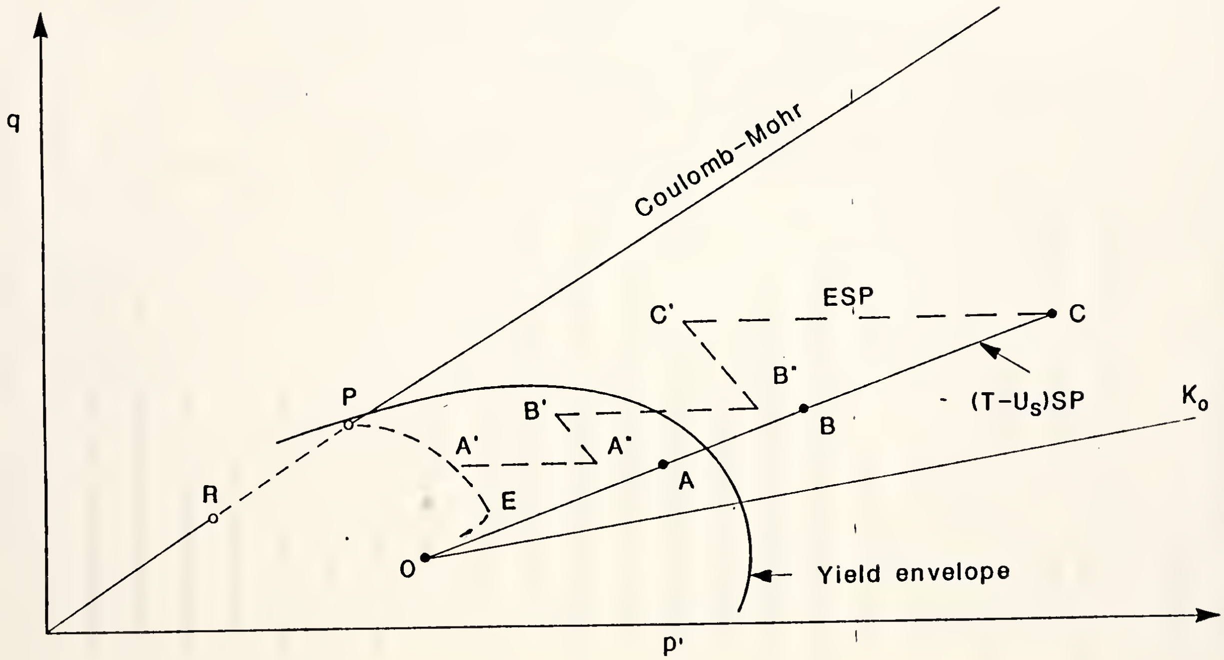


Figure 6.5 Effective Stress Path During Multi-Stage Loading. From Watson et al (1984).

From the above discussion it is clear that stage loading and preloading result in very definite increases in strength. Further, by controlling the effective stress paths so as to eliminate strain softening, it is possible to minimize lateral deformation. For peats and organic soils, stage loading is an effective way of developing necessary strength. One common feature of loading such soils appears to be the development of a mud wave along the edges of the imposed load. This mud wave is possibly due to lateral displacement of the peat and muck and also strain softening. Once a mud wave of this type, or significant lateral deformation, occurs a predictive model no longer applies, unless the model can account for this phenomenon. A study of the literature about embankments on peats reveals that most of the analyses used do not account for this effect. Further, mud waves or heave are often observed during the construction of embankments on such soils. For such a combination of analyses and behavior, it is not surprising that many predictions differ appreciably from observed behavior.

The use of a surcharge for reducing secondary settlements has been recommended by many, for Ex. Johnson (1970). Landva (1980), however, questioned the usefulness of a surcharge in reducing the rate of creep. On the basis of data from a description of the construction of a highway embankment on peat and subsequent observations, he concluded that the surcharge had not significantly reduced the creep rate, and that the only advantage of a surcharge was to delay the settlement due to rebound following removal of surcharge. Samson and La Rochelle (1972) found that in general the duration of rebound was more or less equal to the duration of surcharge application.

Once the surcharge is removed, the subsoil becomes overconsolidated to a degree depending on the surcharge. Hence, the creep behavior of an overconsolidated soil has to be examined. Tavenas et al (1978) examined the creep behavior of a lightly overconsolidated clay. One of the conclusions they reached was that the time dependent behavior of an overconsolidated clay was completely described by the time dependent displacement of the limit state surface. Referring to Fig. 6.2b, it is seen that consolidation under a surcharge load will result in the definition of a new yield surface that is homothetic to the initial surface. If only part of the surcharge is removed, and the stress state lies in Zone 111, the creep rate will depend on the location of the point defining that stress state with respect to the initial and final yield surfaces. In other words, if the stress state after surcharge removal lies close to the new yield surface, the creep rate will be high. If the stress state lies close to the previously defined yield surface, the creep rate will be reduced to a value that corresponds to the original creep rate of the material.

One reason why the creep rate did not reduce for the site described by Samson and La Rochelle (1972) could be that the final effective stress state was close to the yield envelope defined under surcharge loading. In this case the creep rate will be high, and more or less what it was under the stage two loading. An alternative explanation would be that the final stress state falls within Zone IV in Fig. 6.2b. If this were the case, the soil would remain stable at first, but, with time, creep deformations would develop. This would explain the initially stable conditions observed. For stress conditions in this zone, the yield envelope will move from y_1 to y_o. During the process, it will pass through the applied stress conditions and, when this occurs, an apparent creep failure will occur. If this is the case, the embankment built by Samson and La Rochelle (1972) should exhibit some sort of failure, some time in the future.

In general, however, there is good agreement that the effect of a surcharge is to reduce the rate of secondary deformations. The concept has been used and recommended by several people such as Edil (1983) and Gruen and Lovell (1983).

BEHAVIOR DURING AND AFTER CONSTRUCTION

Peats and mucks occur in nature at a high water content, and as previously discussed, have a high compressibility and permeability. Due to the presence of organics, they may also contain a small amount of gas. Further, the water table being high, and settlements being large, part of the fill becomes submerged. As the peat or muck is compressed its void ratio decreases significantly as a result of which the values of various engineering parameters such as C_c, k and c_v change. At times, the strengths are so low that the effect of the fill is to displace material to the sides. All these factors have made the taking of reliable field observations, such as pore pressure measurements, difficult.

This section reviews the behavior of embankments on such soils both during and after construction in terms of settlements and the development and dissipation of pore pressures. The behavior is compared with that of embankments on clay. This has been done so as to determine if the behavior observed on amorphous peats and mucks differs significantly from behavior of embankments on clay, in terms of the phenomena involved.

For clays, there are several methods for predicting the development of pore pressure, for example, Skempton (1954), Henkel (1960), Hoeg et al (1969) and Burland (1971). Tavenas and Leroueil (1980) compiled a series of published comparisons between predicted and observed behavior. In general, they found a tendency to overpredict pore pressures which were greater than 20 kPa. They stated that all natural clays are slightly preconsolidated, and hence initial behavior was characterized by a small recompression index and a high coefficient of consolidation. As the first lifts of the embankment are placed, total stresses are induced causing an immediate build up of pore pressure. Since the clay is overconsolidated, the rate of pore pressure dissipation is high. They calculated the value of $B = \Delta u/\Delta\sigma_1$ and found that the average value was 0.4, which was significantly below the theoretical value of 1 for undrained loading of soft clays. They attributed this reduced value of B to drainage that occurred during construction.

Peats and mucks have a very low preconsolidation pressure, and during the construction process quickly become normally consolidated. However, the in-situ permeability at these stress levels is very high (due to the high in situ void ratio) and the c_v value is also large. Hence, excess pore pressure dissipation occurs very rapidly and, in most cases, complete dissipation occurs in three to four months after construction, even though the material may be normally consolidated. Landva (1980) states that the maximum observed excess pore pressures resulted in a B value of between 0.3 and 0.7. For soft clays the maximum was around 0.9 [Fig. 13 of Tavenas and Leroueil (1980)]. Hence, it is logical that the average B value for peats and mucks during construction be less than the average for clays, which is 0.4. In view of this fact it would be unreasonable that field time for consolidation be predictable from a scaling rule of the form of Equation 4.52, since in the oedometer test the excess pore pressure is equal to the stress increment, whereas in the field this is clearly not so. This could be the major reason for the discrepancy between observed and predicted values for time required for consolidation.

A major consequence of this rapid consolidation is that the effective stress path follows close to the K_o consolidation line. Excess pore pressures beneath the embankment can also cause hydraulic fracturing in the adjacent peat and muck due to the low strength. This in turn could be responsible for the lateral dissipation of pore pressures, which is a phenomenon that is often thought to occur for these materials. At this point it must be remembered that the soils classified as mucks can include a material consisting of 74% clay and 26% organics, with a very low coefficient of consolidation and for which pore water dissipation will not be as rapid or as immediate as the true peats.

For clays, a threshold height of embankment exists above which behavior is normally consolidated. Such behavior is characterized by slow rates of dissipation of pore pressure and by values of B of 1. This type of pore water response does not seem to occur for peats and probably does not occur for mucks either, indicating that the effective stress path for peats and mucks possibly does not follow the yield envelope, but just displaces it further down the K_o line. Lake (1961) and Forrest and MacFarlane (1972) have, however, reported values of 1 for B.

If loading is carried on to failure, strain softening occurs for clays, with the effective stress path moving down the critical state line. This path can only be followed by an increase of pore pressure, and in this case, B_f is greater than one. Values of B_f greater than 1 do not seem to have been reported for peat, even though failures are frequently reported. In other words, behavior of embankments on clay, loaded until it is normally consolidated, is typically characterized by the undrained stress-strain-strength properties of the normally consolidated destructured clay, whereas the behavior of peats and mucks is different from the undrained stress-strain properties of the destructured material. This difference is probably because of the very significant drainage that occurs during loading.

None of the theoretical methods available seem suitable for predicting pore pressure development on such soils. The best approach is to study the variation of the B parameter with depth, for various stages of construction, and to determine typical values of B associated with the various stages of construction of embankments on such soils. Once such typical values are obtained, the excess pore pressure can be predicted, knowing the total stress increment. This semi empirical method was suggested by Tavenas and Leroueil (1980) to predict pore pressure response during those stages of construction on clay, when the soil is overconsolidated. Insufficient data prevent the application of this method to peats and mucks. As the excess pore pressure dissipation during all stages of construction on peats and mucks is rapid and similar to the behavior of overconsolidated clay, it is likely that this method would be suitable for all stages of construction on peats and mucks, i.e. even when these materials are normally consolidated.

The long term pore water response for peats and mucks is quite similar to that of clays. Just as in the case of clays, significant pore water pressures are occasionally reported a long time after construction. This behavior has been attributed by some to creep. It is worth mentioning again that some mucks which have a high clay content may exhibit excess pore pressure behavior quite similar to that of clays.

The deformation behavior can similarly be examined during the various stages of construction. One important factor, however, is that displacement of peats and mucks is a very distinct possibility if there is no root mat or similar type of base reinforcement, and this probably does occur to varying degrees during construction on such materials. Such behavior is not observed to a similar degree for clays and hence comparisons should take this factor into account.

For clays, since initial construction occurs when the soil is overconsolidated, the effective stress path follows approximately the K_o line, and accordingly deformation behavior of the foundation is similar to that of an overconsolidated clay loaded under K_o conditions. For peats and mucks the preconsolidation pressure is very low, and this effect is observed in that a working mat can often be placed without any large settlements occurring. However, this supporting action may also be produced by a root mat.

For clays, as loading continues, the effective stress path reaches the limit state curve for the clay. The clay becomes normally consolidated, and its initial fabric and properties are modified. This results in a change of the stress-strain behavior and further undrained loading results in strain softening, which corresponds to plastic flow. The stiffness of the clay is reduced and the undrained modulus may decrease by a factor of two. Deformations of the foundation soil reflect this reduced stiffness, with settlements being larger than initially, and accompanied by lateral deformations which are of the same order of magnitude as vertical deformations. For peats and mucks, permeability being high, this sort of undrained response does not seem to be very important. Heave and mud waves occur, which may be due to displacement of material directly under the embankment, or due to a sudden flow of pore water to the sides.

Long term settlements of embankments on clays differ from those on peats and mucks in that the rate of secondary settlement is much smaller for clays than for peats and mucks. The secondary settlement rate is much higher than that obtained from laboratory tests, probably because of the existence of stress conditions that differ from those in the oedometer test. This difference is most noticeable for narrow high embankments. These embankments subject the material to a shear stress greater than that applied during the test, thus accounting for the different creep rates.

As regards lateral displacement, Tavenas and Leroueil (1980) observed three distinct types of behavior. Initially the response was that of a drained overconsolidated clay subjected to a stress path close to the K_o line. During this stage the lateral displacements would be relatively small and represent only a small fraction of the settlement. Lateral deformation in this range would also increase linearly with load. Towards the end of construction, as the clay becomes normally consolidated in significant parts of the foundation, an undrained response occurs. During this stage the lateral response would reflect the reduced rigidity of the normally consolidated clay, by increasing more rapidly with the imposed load and being more or less equal to the increment in vertical settlement.

After construction of the embankment, as consolidation occurs, behavior should be typically drained, with lateral displacements being less than vertical. Analyzing available observations Tavenas and Leroueil (1980) developed two equations to predict lateral deformations during construction. For initial responses the relationship between vertical and lateral movements was:

$$\Delta y_m = (0.18 \pm 0.09)\ \Delta s$$

and, for the undrained response the deformation was

$$\Delta y_m = (0.91 \pm 0.2)\ \Delta s$$

where Δy_m was the maximum lateral displacement and Δs is the vertical settlement. For peats and mucks, almost no data are available about lateral deformations, apart from the fact that mud waves and heave are quite occasionally observed. Regarding long term lateral deformations, no data seem to be available.

As regards behavior under traffic loads, Lea and Brawner (1959), Lea (1964) recommended a minimum of 1.2m of fill for secondary and primary highways, and 1.4 to 1.5m for freeways. The fill should be placed on material that has been preconsolidated under a load equal to that of the embankment plus the maximum expected traffic load. Landva (1980) recommended the use of corduroy to spread the load. As a result of this load spreading action the corduroy prevented any significant deterioration of the road. Landva (1980) also concluded that the rigidity of the base course was important. The deformation and settlement of a 1.2m non-compacted fill, on double fabric which was separated by a thin layer of sand, under wheel loads, was much greater than that of a 0.4m compacted fill on corduroy. Also a comparison between the behavior of a 1.2m and a 0.6m fill of the same type and on the same subsoil revealed no major difference in the behavior under loading. This led him to conclude that the rigidity was more important than the thickness of the fill and that this rigidity was considerably enhanced by base reinforcement.

Keyser and LaForte (1984) studied various embankments on different types of peats 8 years after construction. They concluded that though the riding quality over the peat was good, the pavement surface was 50% rougher over the peat deposits than over the no peat areas. The riding quality of pavements in transition zones between peat and no-peat sections was about 40% rougher than no-peat areas. Transverse cracking was not influenced by the presence of peat deposits, while on the other hand the number of longitudinal cracks was 2 to 4 times higher than in other areas, and increased with time. They were not able to correlate the length of horizontal cracking with either the height of fill, or the depth of the deposits. Though their design was based on the concept of preconsolidation prior to paving, they did not use a surcharge.

From the description presented above it is clear that for embankments on peat and mucks, measurements of the pore pressure response and lateral deformation are not commonly made. As a result, much important information is not available. Effective stress paths too cannot be determined. Still, it appears that an undrained loading condition with slow excess pore pressure dissipation probably does not occur, since considerable drainage occurs even during construction itself.

FAILURE MODES

In this section the failure modes of embankments on such materials is examined. Peats and mucks have low effective unit weights. Consequently, if they exist over clays, the clays will be very soft. Landva (1980) showed that the stress transferred to the subsoil for a typical embankment which exerted a load of 30 kPa on the peat, would be around 20-25 kPa. This could cause failure, if the clay were very soft. Lea (1964) measured the vane strength of a 6m clay layer lying under a 6m peat layer and found it to be between 6-16 kPa. He stated that on the basis of his experience, that when embankment stability was a problem, it was fairly certain that there was soft clay beneath the deposit. Lea and Brawner (1959) stated that where highway distress was assumed to be due to the peat, the real cause could be traced to the soft soil immediately underlying the peat. This, they said, was true regarding both consolidation movements and embankment stability. MacFarlane and Rutka (1959) examined road embankments on about 50 different muskeg areas and concluded that the cause of the shear failures in muskeg areas underlain by a soft mineral soil was not the peat layer itself, but the layer of soft soil. Landva (1980) concluded that failures of embankments on peats over soft subsoil was not uncommon, and also, that such failures never seemed to occur for peat over firm subsoil.

The exact mechanism of failure occurring for peats over soft subsoil is not known, but Landva (1980) concluded that both spreading and rotational failures are possible. He based this conclusion on observed behavior and the fact that the strength of the soft clay was low.

For amorphous peats and mucks a rotational failure wholly contained within the peat or muck layer, or extending into the peat layer is possible. Since consolidation in the peat or muck usually proceeds rapidly, it is likely that the failure is initiated in the weak clay sublayer, and progressively travels through the entire weak, and as yet unconsolidated clay layer and later, through the peat layer. For peats and mucks on firm subsoils, however, such an initiation of failure is not possible, thus indirectly confirming that this could possibly be the mechanism behind a rotational failure for the case of embankments on peats and mucks underlain by soft clays.

Another type of failure that is possible is a spreading type failure. Landva (1980) postulated a mechanism for this type of failure based on descriptions of actual failures. For this type of failure only the upper part of the clay would be involved. As construction proceeds, pore pressures increase both along the center line of the embankment and at the interface between the peat or muck and the soft clay. This could result in a progressive failure along the interface which would ultimately culminate in a spreading type failure. This type of failure is quite capable of being analyzed by programs such as the BLOCK option in STABL4. Spreading of the soil could cause a failure along the center line of the embankment. As spreading occurs, passive pressure gradually builds up and further sliding is arrested. This type of lateral movement of the peat or muck appears to be quite similar to the behavior of clay when normally consolidated, with the effective stress state on the yield locus.

One other type of failure occurs as a result of improper widening of an embankment. When an existing embankment is widened with the possible raising of the existing grade, the additional pressure imposed is as shown in Fig. 6.6. As a result of this stress increment, settlements occur that will more or less be proportional to the applied pressure. Hence the newer shoulders of the embankment will settle the most. This causes a stretching of part of the upper portion of the embankment and can cause longitudinal cracking. Occasionally, the situation is further worsened by adding more material to bring the surface back to grade.

Hanrahan (1964) describes the widening of a road in Ireland. The mat was removed and gravel placed. This results in progressive local compression, slips, upheavals, ridges, hollows, etc. These depressions are corrected by adding extra material. This procedure solved the problem only temporarily. Finally, along portions of the road, part of the gravel was replaced by lighter material - peat bales. Long term observations showed that those sections where peat bales were used performed satisfactorily while the sections with gravel progressively deteriorated. The various stages of deterioration of the road are shown in Fig. 6.7.

Landva (1980) states that widening of embankments on such soils can easily lead to differential settlement and cracking as a result of the differential loading. In extreme cases local

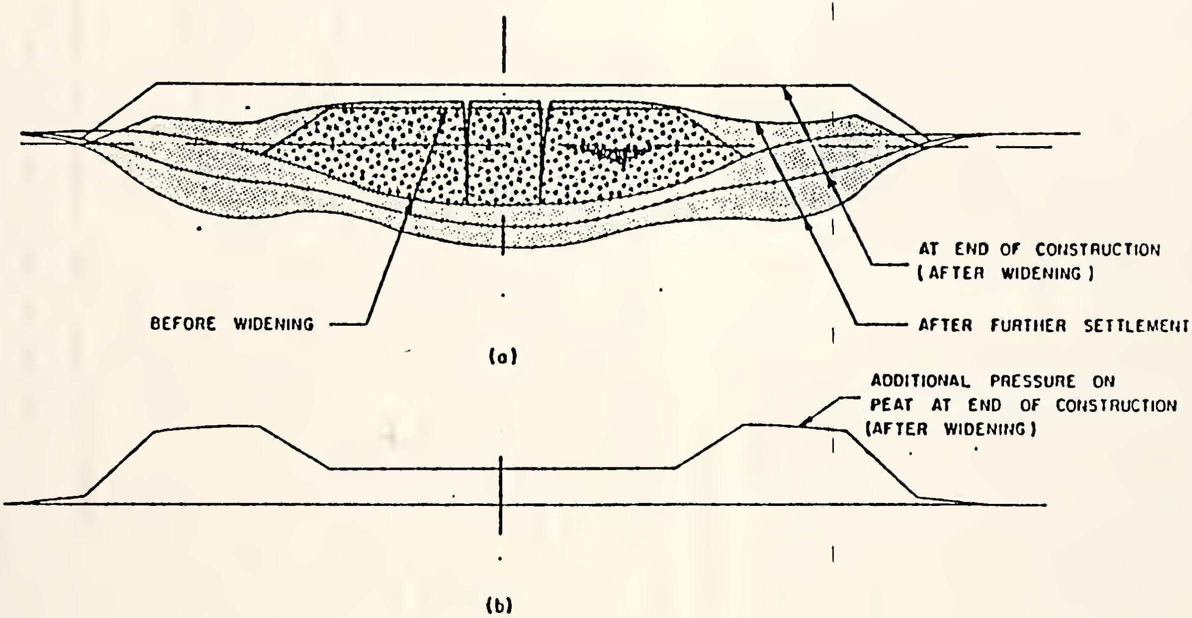


Figure 6.6 Embankment on Peat Before and After Widening.
From Landva (1980).

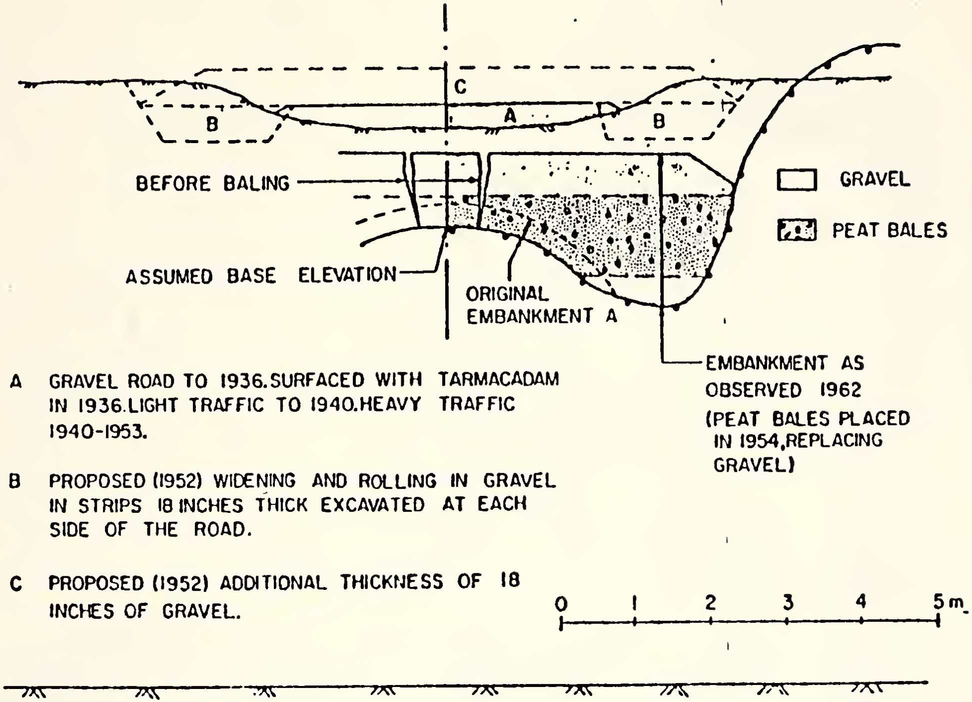


Figure 6.7 Section Through Road on Peat. From Landva (1980).

shear failures, slips, etc. may be observed as shown in Fig. 6.6. In such cases, use of a lightweight fill along the edges will result in a more uniform application of pressure.

THE EFFECTIVE STRESS PATH AND YIELD ENVELOPE CONCEPT APPLIED TO PEATS AND MUCKS

Based on the description of embankments on peats and mucks, a qualitative model of behavior using the effective stress path and yield envelope concept can be formulated. It is assumed that these soils have a high in situ permeability, void ratio and consequently a high coefficient of consolidation. Also, it is assumed that the $K_{o_{NC}}$ value of these soils is a constant, independent of the consolidation pressure. Thus, from Fig. 6.8, the initial condition of the soil is represented by the point ′0′. As the loading occurs, the effective stress path follows approximately the K_o line. Since the quasi-preconsolidation pressure of these soils is quite low, it is very likely that the stress path moves along the yield locus and hits the critical state line resulting in strain softening behavior. This is indicated by the effective stress path 0′-1′-2′-3′. The rate of consolidation being high, the stress path moves horizontally, beyond the original yield envelope and a new yield envelope is defined. This proceeds until finally the stress level is high enough so that the effective stress level never reaches the critical state line. At this stress level, a typical stress path would be 4′-5′-6′-7′-8′-9′. This would mean that the initial

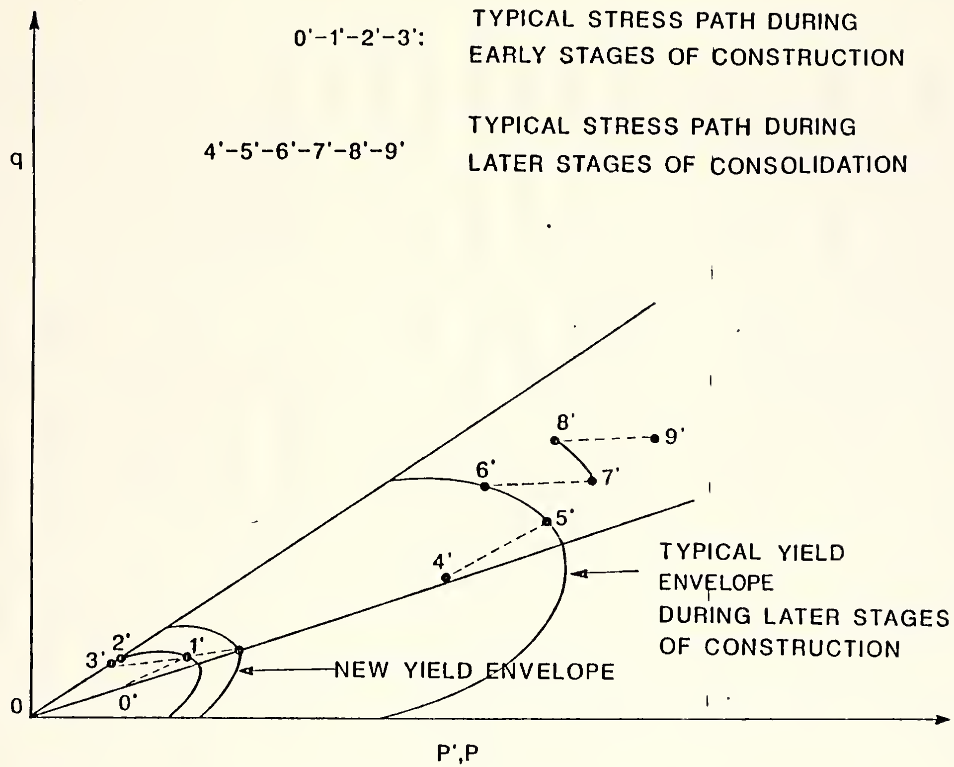


Figure 6.8 Proposed Effective Stress Path/Yield Envelope Model for Peat and Mucks.

loading would have to be done in small increments to prevent excessive strain softening behavior. It would also confirm the observation that for firm subsoils no failure will occur, since the effective stress path at higher stress levels does not reach the critical state line on account of the high coefficient of consolidation. This high coefficient of consolidation also results in inconsiderable drainage taking place during construction.

For the firm subsoil the stress state does not reach the critical state line and strain softening does not occur. For soft clayey subsoils below the peat or muck, however, the situation is quite different. Here, increase in stresses due to loading will cause the effective stress path to reach the yield locus and, after moving along it to the critical state line, to start tracking down the critical state line. However, the permeability of the clays being usually several magnitudes lower than the muck or peat, consolidation and definition of a new yield locus does not occur as rapidly as for the peat or muck. As a result, while further stage loading results in safer conditions for the peat or muck (due to the high rate of consolidation and almost immediate definition of the yield locus) for the clays, the conditions become worse until finally strain softening and plastic failure occur.

For stiff clays, on the other hand, the current yield locus is not as close to the existing stress state and consequently this behavior is not seen. It must be repeated that mucks with a high clay content and low in-situ permeability would not behave according to the model presented above, but instead would behave more like a soft clay.

PREDICTIVE METHODS OF ANALYSIS

This section deals with the purely predictive methods of analysis available. For an analysis to be of use, it must model all the basic aspects of the phenomena. Even for clay foundations the prediction capability is not very good as regards both pore pressure development and settlements. The results of the "Foundation Deformations Prediction Seminar" held at MIT (1975) showed that the ability to predict embankment behavior on even clay is questionable. The situation appears even worse for peats and mucks.

One typical and illustrative example is the predictions made by Sarac and Popovic (1981) for an embankment on an organic soil layer. They performed 5 types of analyses of varying degrees of sophistication. The first analysis modeled the heterogeneous layer with a nonlinear e-σ' relation and a varying value of the permeability. The second analyses was the same as the first except that the e-σ' relation was taken to be linear. The third analysis was the same as the first, except that k was kept constant. The fourth was the same as the first except that the soil was modeled as a homogeneous material with properties corresponding to the middle of the clay layer, while the fifth was the same as the first except that the thickness decrease during consolidation was neglected. Initial undrained settlements were calculated for the plane strain case using the solution for a finite elastic layer with a uniform strip load.

They concluded that the influence of a nonlinear $e-\sigma'$ relationship, a varying permeability coefficient, layer heterogenity and thickness decrease are very small. Even though the analyses differed significantly from each other, far more important than sophistication of analyses was, they said, the statistical evaluation of a large number of results, as the scatter of their test data was quite significant. They also noticed that observed settlements during the initial period of consolidation differed considerably from the calculated ones.

The reason for this could be that the effective stress path followed during construction was not the path implied in common types of settlement analyses. When the stress state is moving along the yield locus, the response is similar to undrained loading. Further, when the stress state is moving down the Mohr-Coulomb line, strain softening and plastic failure is occurring. Both these types of deformation are not modeled, and hence cannot be accounted for in conventional types of analyses. Another problem is the uncertainty regarding the permeability and also the exact boundary conditions.

In view of this, purely predictive methods are to be viewed as useful only in providing an idea of the order of settlements to be expected. For peats and mucks, if the model proposed initially is acceptable, for early stages of loading the effective stress state will move along the yield locus and later the critical state line, following a stress path quite different from that implied by conventional analyses. This could be why the settlements observed during the early stages of consolidation (when the stress path implied by the analysis differs considerably from the actual stress path) are quite different from those predicted. For later stages of consolidation the actual and implied stress path will not differ as much. Another factor that is not considered in the analysis, but which occurs, is the displacement of the soft and at times almost slurry like peat or muck by the fill. This would result in greater settlements than those predicted.

From the above discussion it is clear that for peats and mucks, sophisticated predictive methods may not be of much use if important effects like movement of the effective stress path along the yield locus, strain softening and displacement are not taken into account. In such cases simplified analyses of the type reviewed by Gruen and Lovell (1983) are likely to be as good, providing a rough indication of the performance to be expected. If the effective stress path can be made to follow the theoretically implied stress paths, by monitoring pore pressures and controlling loading, then the methods described in Chapter IV should give a good picture of behavior to be expected, provided no displacement of the material has occurred.

USE OF BERMS, DRAINS AND LIGHTWEIGHT FILLS

Berms, drains and lightweight fills are often used for problem materials. In this section the use of each will be analyzed. It will be seen that they are useful in certain situations, but not always.

Berms are used often on soft soils to reduce deformations and to increase the safety margin for borderline stability problems. Landva (1980) concluded that for fibrous peats, berms contributed little to embankment stability, the fibers in the peat providing sufficient internal lateral support to prevent lateral deformation. For amorphous peats and for mucks the situation is quite different, especially if the underlying soil is soft, for example: marl, soft clay, etc. In such cases berms play a useful role. Raymond (1968,1969) and Hollingshead and Raymond (1971,1972) approximated such soils as being elastic, and showed that berms had a strong beneficial effect. The use of berms reduced the occurrence of undrained movements, limiting the magnitude of shear deformations. Raymond (1968) showed that if berms were to be used, the sequence of construction played a role. Construction from the outer edges inwards (simultaneously from both sides) imposed lesser shearing stresses than when constructing from the center line to the sides or from one side to the other. Moving from the outsides to the center also would trap any mud wave in the center. In other words, for amorphous peats and for mucks, berms have a beneficial influence as regards settlement, bearing capacity and stability, especially if the underlying soils are weak.

Vertical drains are not often used in peats on account of these relatively high permeabilities. The same holds for mucks, except that clayey mucks undoubtedly exist for which the in-situ permeability is low enough to make the use of drains attractive. Lea and Brawner (1959), Hillis and Brawner (1961), Lake (1961) studied the use of sand drains and concluded that they were not beneficial, from the standpoint of pore pressure dissipation, for peats. Weber (1969) reported that sand drains increased the stability of the fills not for reasons of reduction of drainage length, but because of the pile action of sand drains. For clayey peats (mucks) drains were beneficial. If the underlying material is a soft clay, drainage will be required. If drainage of the peat or muck and underlying soft soil is ensured either naturally or through drains, there will be little difficulty using stage construction in attaining the required height of fill. Sometimes a peat layer may exist between two impermeable layers. In such a case a drainage route for the unconsolidated material must be provided. Landva (1980) concluded that sand drains also resulted in reduced wave action under traffic loads and suggested that the drains might be carrying some load directly or through arching. As regards drainage, the drains would become clogged as well as surrounded by peat of reduced permeability. It should be noted that the beneficial aspects of sand drains appear to derive mostly from the pile-like action of the sand columns. Band drains are thus unlikely to be useful except possibly for any underlying soft soils of low permeability.

Lightweight fill such as sawdust, wood waste, compressed bales of peat, etc. have been used for overcoming settlement problems, for ex. Hanrahan (1964). By reducing the load on the peat or muck, settlements are reduced. The subbase of the embankment can be constructed of the lightweight fill. Ideally, to prevent decomposition, the fill should not extend above the groundwater table. This method, however, is generally used only for minor roads.

CONCLUSION

The behavior of an embankment on peat or muck can be viewed in terms of the effective stress and yield envelope concept, and a comparison made with the behavior of embankments on clay. Such a comparison reveals that case histories of embankments on peats and mucks do not make use of this effective stress concept, though drainage during construction occurs to a far greater extent than for clays. Preloading and stage loading were found to be useful for these materials. Failure models commonly observed are rotational failures and also spreading failures at the interface with the underlying soil. Failures seem to occur only when the underlying soils are weak. The effective stress path/yield envelope concept was applied to peats and mucks and a qualitative model developed. Undrained movements, strain softening and lateral displacement of the peat and muck were suggested as being the main reason for the lack of success of the purely predictive methods. Finally, the use of berms, drains and lightweight fills was studied and it was found that each was useful under certain conditions.

CHAPTER VII - A DESIGN METHODOLOGY

INTRODUCTION

From the preceding chapters it is clear that undisturbed sampling of peats and mucks is not easy. Correct testing of this material is also difficult, with special equipment such as a K_o triaxial cell being required. Another factor is that these materials are highly variable. In view of these facts, detailed and accurate determination of soil parameters representative of the entire deposit for normal engineering projects is not easy. Apart from this is the fact that the analyses presented earlier [Berry and Poskitt (1972) and Mesri (1977)] are formulated as initial boundary value problems of the heat conduction type. Apart from the problems mentioned earlier about accurately determining the values for the material properties the problem of determining the boundary conditions is also involved. This is particularly difficult in the case of multi layered soils - Ex: peat underlain by soft clay.

The above reasons also make it difficult to predict accurately the bearing capacity. For peats and mucks, measuring the undisturbed shear strength is not practical for most projects. Further, Bjerrum (1972) has shown that there have been many examples of base failure, in spite of the fact that the design factor of safety was more than 1.0. It is understood that considerable uncertainty exists in determining the values required for an analysis, the experience of the engineer in charge being relied on to achieve a safe design.

An alternative approach is to monitor the field behavior of the embankment foundation system during construction to detect any sign of impending failure, and to predict future performance of the embankment. This observational procedure was formalized by Peck (1969), and later revised by others - Sekiguchi and Shibata (1979), Asaoka and Matsuo (1980). The advantage of this method is that the design is based on the observed behavior of the entire soil system, and not on the behavior of a few disturbed samples. The observational methods to be discussed were initially formulated for soft clays, but, are particularly suited to peats and mucks.

This chapter presents three observational methods, one to determine the loading rate and ultimate bearing capacity. The other two are to be used for surcharge application to determine how long the surcharge should remain in place to minimize future settlements. Finally, a complete design methodology for embankments on peats and mucks will be presented.

BEARING CAPACITY

From the effective stress path/yield envelope discussed initially it is seen that during construction, response is under conditions of partial drainage. Sekiguchi and Shibata (1979) did several finite element analyses of undrained response under plain strain conditions using an elasto-visco plastic model. They assumed that the soft clay beneath the embankment behaved elasto-visco plastically, under undrained conditions, when subjected to an embankment load. As seen from the experimental results, the undrained response of materials with organic contents as high as 40-50% is quite similar (with the exception of pore pressure development) to soft clays. Hence, it is felt that the undrained response of such materials too can be modeled by an elasto-visco plastic model. In fact, the model, on the basis of the observed similarity in behavior between peats and mucks, would apply to peat. In other words, the conclusions of Shibata and Sekiguchi (1981) would apply to peats and mucks as well.

For reasons of practicality, the indicators of impending embankment failure must be easy to observe. Normally, the parameters observed are the surface settlement ρ_2 along the embankment center line, excess pore water pressure u_e at the center of the embankment and the outward horizontal displacement δ at the toe of the embankment. Fig. 7.1 shows the definition of the various symbols used. The measurement of these quantities

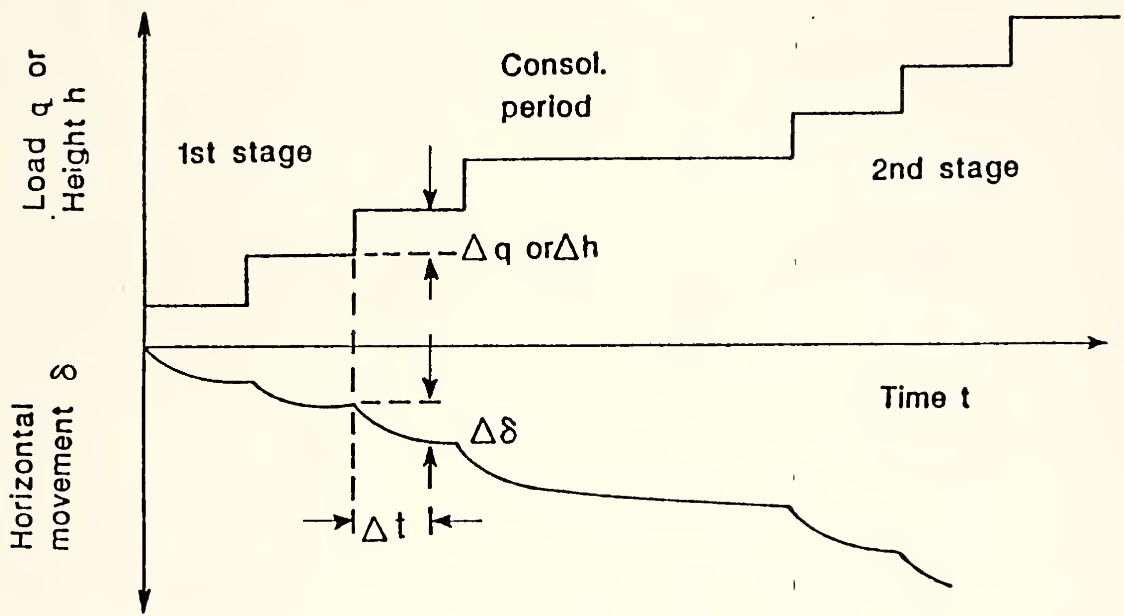


Figure 7.1 Definitions of Symbols Used. From Shibata and Sekiguchi (1981).

for peat and muck must be accomplished using appropriate instrumentation, taking into account the fact that strains will be large.

If embankment loads are considered as being applied in a step-wise procedure, consisting of an instantaneous application of load Δq, followed by a rest time Δt, the loading rate $\dot{q} = \Delta q/\Delta t$ can be defined. Also, if the increment of the lateral displacement $\Delta\delta$ corresponding to the load increment Δq is known, the load deformation modulus $\Delta q/\Delta\delta$ can be defined. Fig. 7.2 shows a plot of $\Delta q/\Delta\delta$ versus the increased load intensity q. As can be seen, when the factor of safety is less than 1.4, the load deformation modulus decreases linearly with the applied load. If this decrease were true, the immediate ultimate bearing pressure would be given as the value of q at $\Delta q/\Delta\delta$ equal to zero. Sekiguchi and Shibata (1979) felt that $\Delta q/\Delta\delta$ rather than $\Delta q/\Delta\rho$ was of more use in predicting the immediate ultimate pressure because, under partial drainage, the outward horizontal movements at the embankment toe show a visco plastic flow of the foundation soil, unmasked by any consolidation settlement. The data from pore pressure and vertical settlements did not indicate any similar sign of impending failure. Further analysis by Shibata and Sekiguchi (1981) showed that this linear decrease of $\Delta q/\Delta\delta$ vs. q was valid under conditions of partial drainage. They also found that the ultimate undrained bearing capacity increased with a decrease in the loading rate, and with an increase in k, the coefficient of permeability. This is in conformance with the

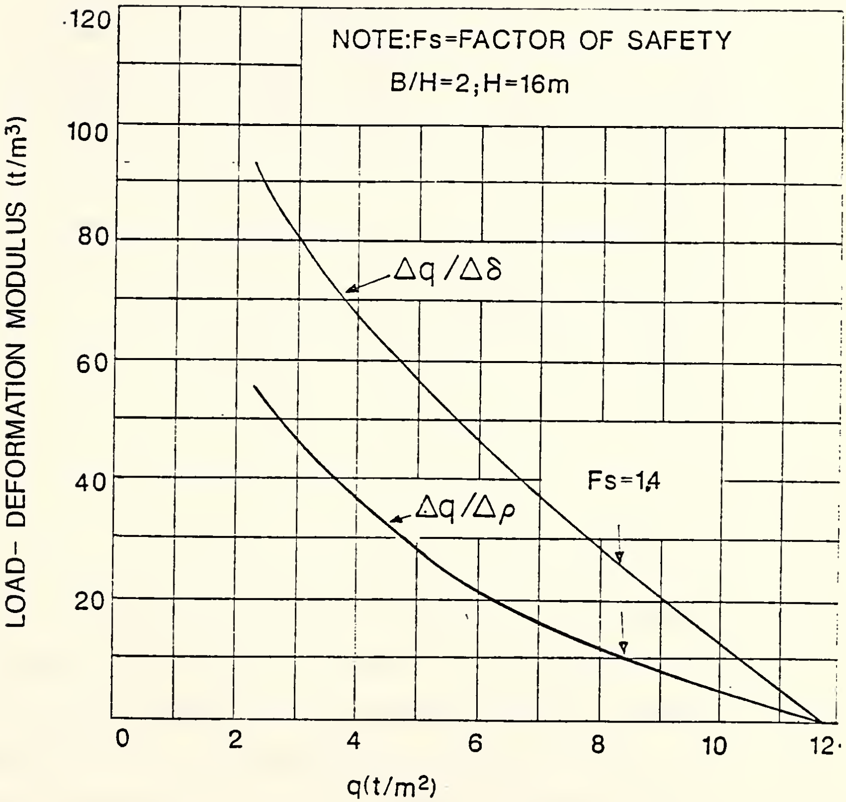


Figure 7.2 Relation Between Load-Deformation Modulus and Load-Intensity. From Sekiguchi and Shibata (1979).

effective stress path/yield envelope concept presented earlier, as a slower loading rate and a higher permeability allow for some drainage to occur, resulting in a movement of the current effective stress point away from the failure envelope.

Shibata and Sekiguchi (1981) applied this method to data from three test embankments. Details of the embankments and the subsoil conditions are shown in Table 7.1, and the results are shown in Figs. 7.3, 7.4, and 7.5. Instead of plotting $\Delta q/\Delta\delta$ vs. q they plotted $\Delta q/\Delta\delta$ vs. the fill height 'h'. Fig. 7.3c shows that the behavior predicted by the model is followed, the same pattern being observed for each of the embankment construction stages. It also shows the benefits of stage loading in increasing the bearing capacity. Fig. 7.4c shows that though driving sand compaction piles increased the initial lateral deformation, subsequent lateral displacement was reduced and the ultimate bearing capacity was increased. Fig. 7.5c shows the application of the method to data from the Aiko test embankment. This embankment was founded on a muck and its purpose was to evaluate sand drain performance. It is seen that the sand drains did not reduce the lateral deformation and did not increase the bearing capacity. Some mucks may have a permeability similar to soft clays and consequently, may require drains. Shibata and Sekiguchi (1981) plotted $\Delta q/\Delta\delta$ vs. B/D_f, where B is the base width of the embankment and D_f is the maximum depth to an observed sliding surface, as shown in Fig. 7.6. On the basis of these data, they tentatively suggested that loading be stopped

Table 7.1 Parameters of Three Test Embankments.

SITE	SOIL	DEPTH (m)	w (%)	C_H (kPa)	FILL h(m)
SHIRAKAWA	COHESIVE	7-11	40-160	5-25	6-8
KUROSAKI	COHESIVE	7	50-400	10-40	6-7
AIKO	COHESIVE	3 7	50-250 300-700	5-30	4-5

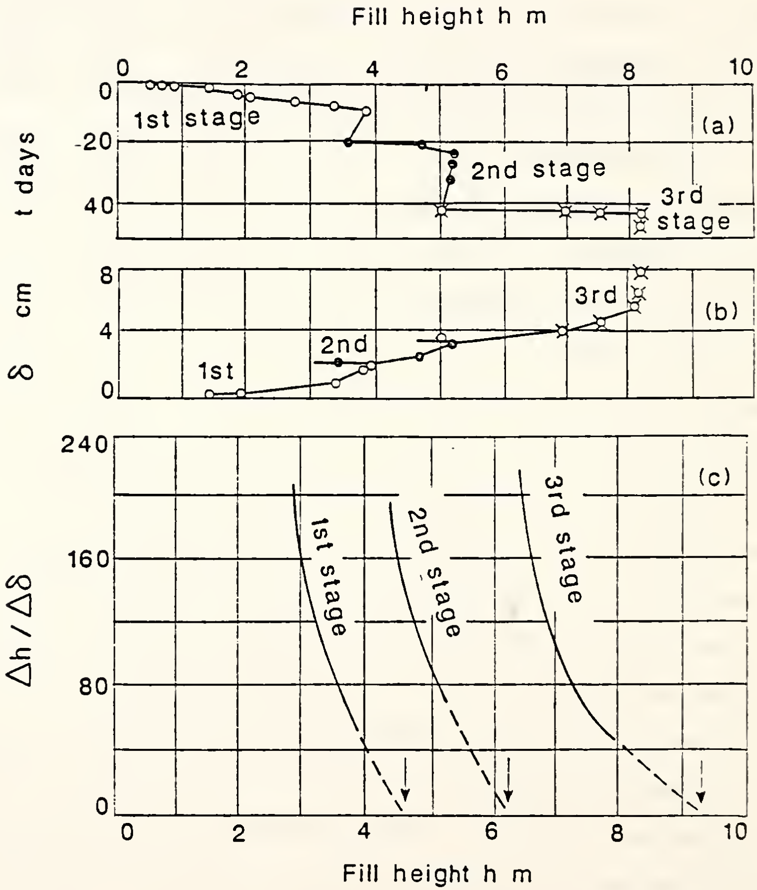


Figure 7.3 Results from Shirakawa Test Embankment. From Shibata and Sekiguchi (1981).

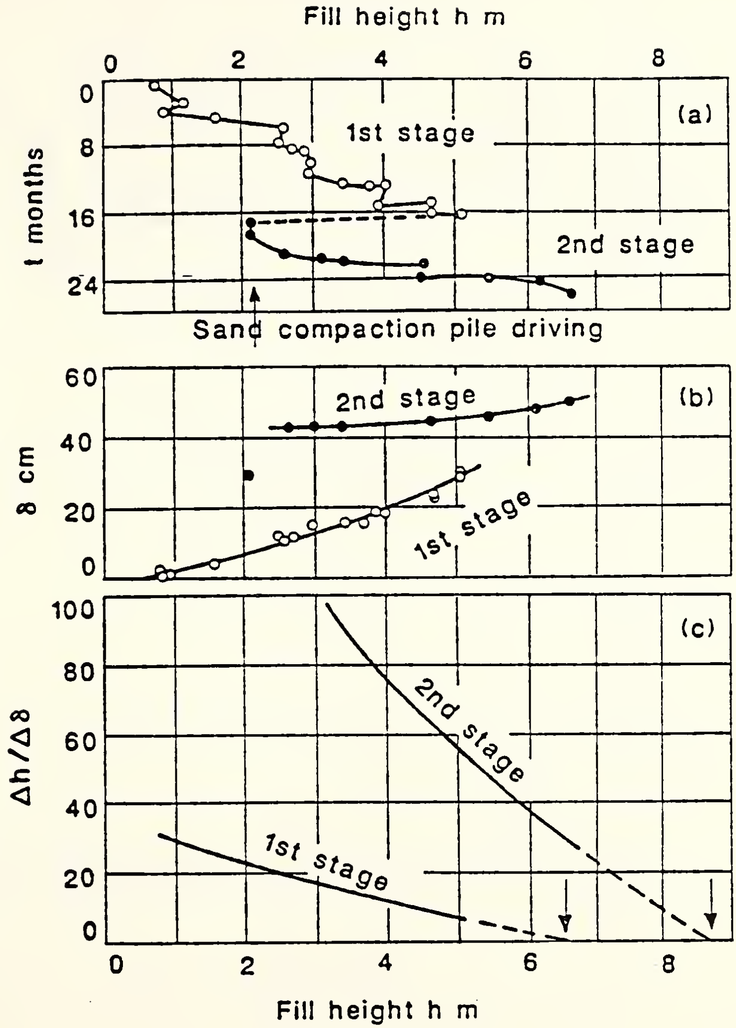


Figure 7.4 Results from Kurosaki Test Embankment. From Shibata and Sekiguchi (1981).

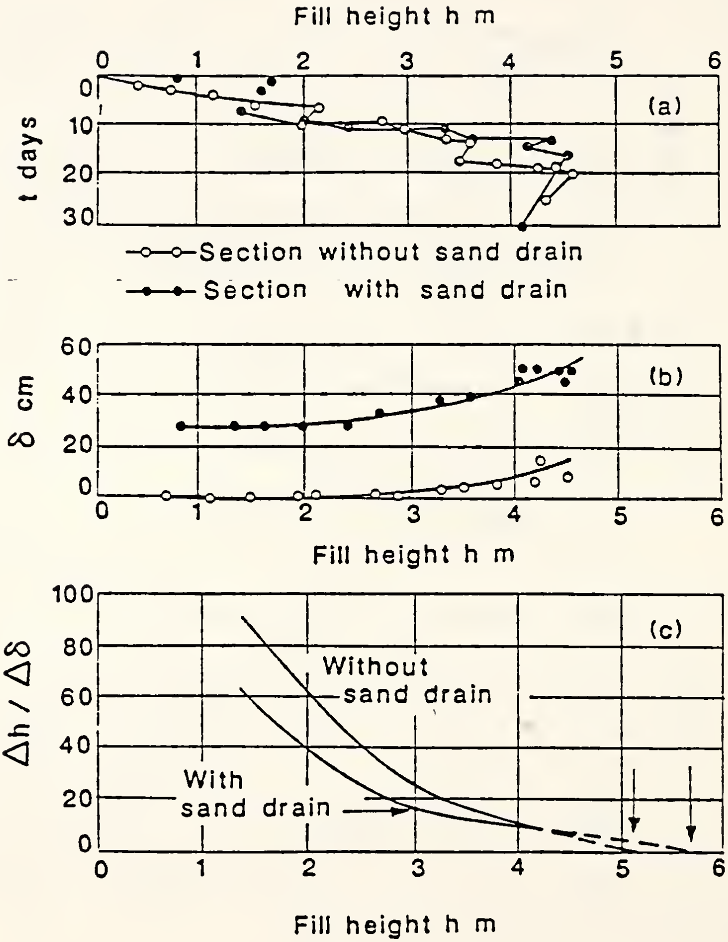


Figure 7.5 Results from Aiko Test Embankment. From Shibata and Sekiguchi (1981).

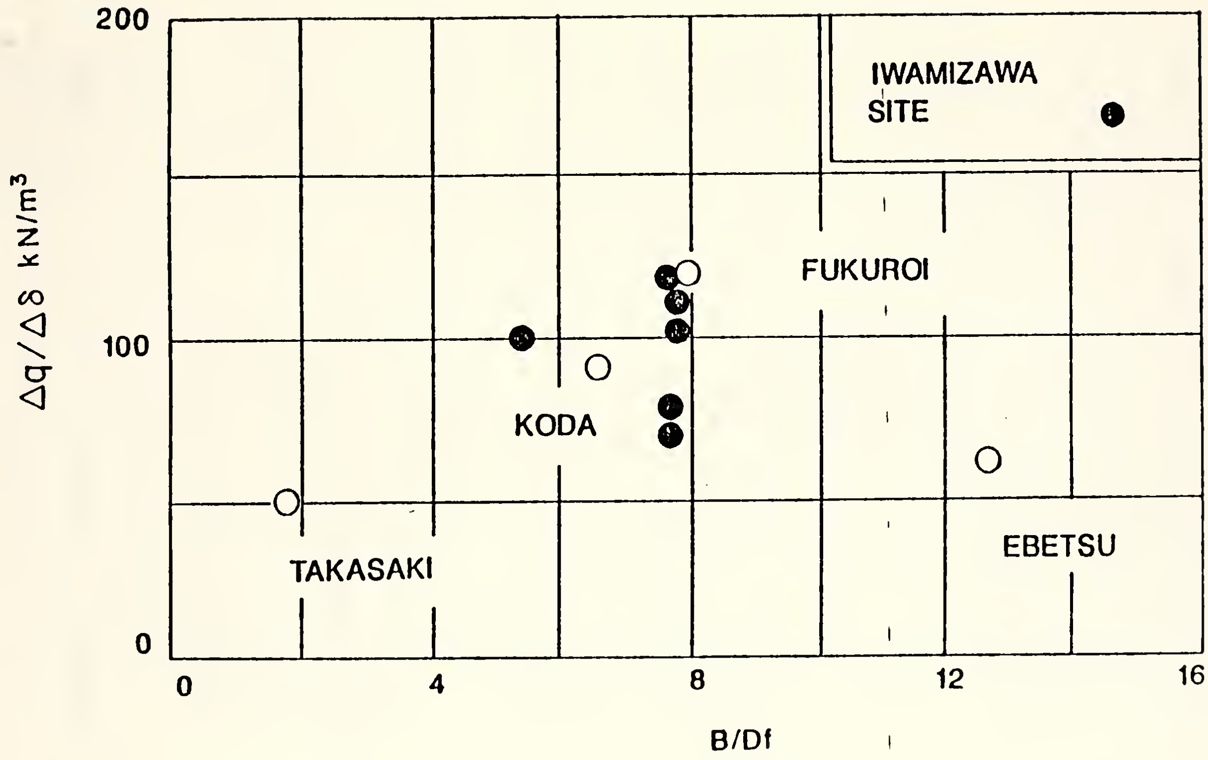


Figure 7.6 Lateral Deformability Factors in Critical Conditions. From Shibata and Sekiguchi (1981).

when $\Delta q/\Delta\delta$ had decreased to around 200 kN/m^3. After a suitable halt time, loading could be renewed, making sure that $\Delta q/\Delta\delta$ remained at over 200 kN/m^3.

SETTLEMENT PREDICTION

From earlier chapters it is clear that since peats and mucks are weak and highly compressibly, large settlements are to be expected. Further, these settlements continue, even after excess pore water pressure is dissipated. For reasons previously discussed, preloading strengthened the soil and reduced long-term compression under the design load. When a surcharge is used, it is important to know how long the surcharge should remain in place, to reduce the settlements likely to occur under the design load. Two methods are suggested below. One is the Gibson-Lo approach, while the other is due to Asaoka and Matsuo (1980).

The Gibson-Lo approach is based on the model shown in Fig. 4.9 and suggested by Gibson and Lo (1961). Lo et al (1976) used it to analyze a fill placed on a deposit of mostly clay, Edil (1983) used it to successfully control settlements of a structure on fibrous peat, and Gruen and Lovell (1984) analyzed an embankment on peat using the model.

Briefly, the model is as shown in Fig. 4.9, except that the rheological parameters are assumed to be linear. For large values of time, the time dependent strain $\varepsilon(t)$ may be written as:

$$\varepsilon(t) = \Delta\sigma \; [a + b(1 - e^{-(\lambda/b)t})] \qquad (7.1)$$

where t is time since loading was completed, $\Delta\sigma$ is the stress increment and a, b and λ are the empirical parameters corresponding to the springs and the dash pot in Fig. 4.9. These parameters can be estimated initially from laboratory tests. This is done by differentiating Eq. 7.1 with respect to time from which:

$$\frac{\partial\varepsilon(t)}{\partial t} = \log_{10} \Delta\sigma\lambda - 0.434\,\frac{\lambda}{b}\,t \qquad (7.2)$$

Hence, by plotting the laboratory data in terms of strain rate vs. time, a straight line is obtained. From the slope and intercept of this line, the various parameters can be calculated, and used to obtain an idea of the magnitude of the surcharge load required, as well as the duration of loading. Once the surcharge is placed the parameters are calculated as explained, except in this case the settlement data are obtained from field observations. Using these parameters and Eq. 7.1, the expected strain for the lifetime of the embankment is calculated, and the surcharge is removed when the settlements exceed this calculated value.

The Gibson-Lo model was first formulated so as to account for creep. Since peats and mucks exhibit creep, it is, as shown in Chapter V, useful in predicting behavior at large values of time under loading. Further details regarding the Gibson-Lo procedure can be found in Gruen and Lovell (1983). It should be mentioned however, that the Gibson-Lo model is strictly applicable only under conditions of one-dimensional loading, i.e. where deformations due to shear and lateral displacement of underlying soil do not occur.

Asaoka (1978) showed that in general, settlement could be predicted using the equation:

$$\rho_j = \beta_o \pm \sum_{s=1}^{n} \beta_s \rho_{j-2} \qquad (7.3)$$

where

β_s, s=1 to n are coefficients;

ρ_j is the settlement at time t_j;

$t_j = \Delta t$, j=0,1,2; and

Δt = a constant time interval.

For n greater than 3, statistical identification of the parameters is difficult. However, with n=3, a wide range of problems such as creep, sand seams, sand drains, etc. can be taken into account. If long term observations are possible, the model with n=1 gives good results. In this case:

$$\rho_j = \beta_o + \beta_1 \rho_{j-1} \qquad (7.4)$$

and a plot of ρ_j vs. ρ_{j-1} gives a straight line with intercept β_o and slope β_1. Since, if $\rho_1 = \rho_{j-1}$, settlement is complete, extending this line will cause it to cut a line passing through the origin with a slope of 1:1 at the final settlement. Asaoka

(1978) applied this method to an embankment on a complex soil profile as shown in Fig. 7.7. Sand piles were driven through the soil to a depth of -13m to facilitate drainage. Fill was placed in two stages with a gap of six months in between, resulting in the two lines shown in Fig. 7.7.

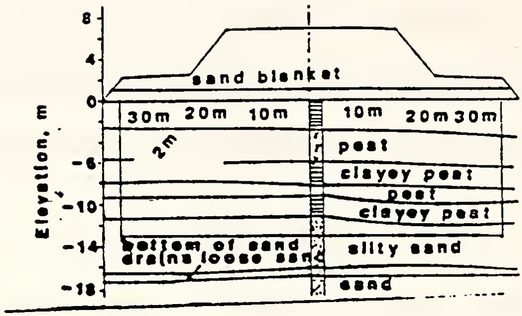
The parameters required for Asaoka's model are shown in Fig. 7.8. Once they are identified, they are used in the equation:

$$\rho_j = \frac{\beta_o}{1-\beta_1} - \{\frac{\beta_o}{1-\beta_1} - \rho_o\} (\beta_1)^j \tag{7.5}$$

to predict future settlement. Orleach (1983) showed that deviations from the graphical behavior described by Asaoka could occur as shown in Fig. 7.9. In this case, higher order models would apply. Matsuo and Asaoka (1981) recommended the use of the higher order models in the absence of long-term observations. Their procedure, adapted for surcharge, is as follows. When the design load is reached, further placement of fill is halted and settlement observations are taken until the final settlements as calculated from the model, where n=2 and n=3, are more or less equal. When this stage is reached, sufficient data are available to predict settlements at any future time. For peats and mucks, this period could range from three to six months.

Statistical identification of the parameters is as follows for the n=2 model. The model in this case is given as:

$$\rho_j = \beta_o + \sum_{s=1}^{2} \beta_s \rho_{j-s}$$



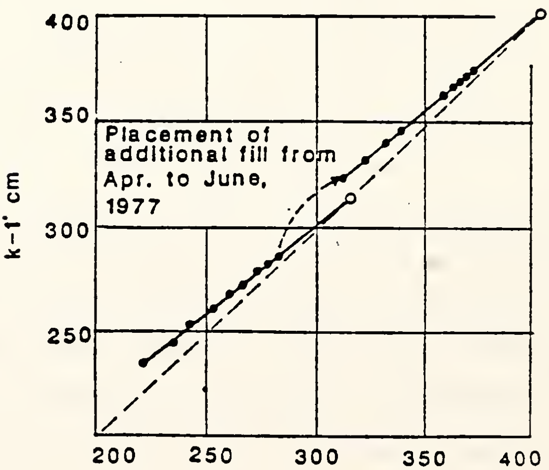


Figure 7.7 Application of Graphical Method to Iwamizawa Test Fill. From Asaoka (1978).

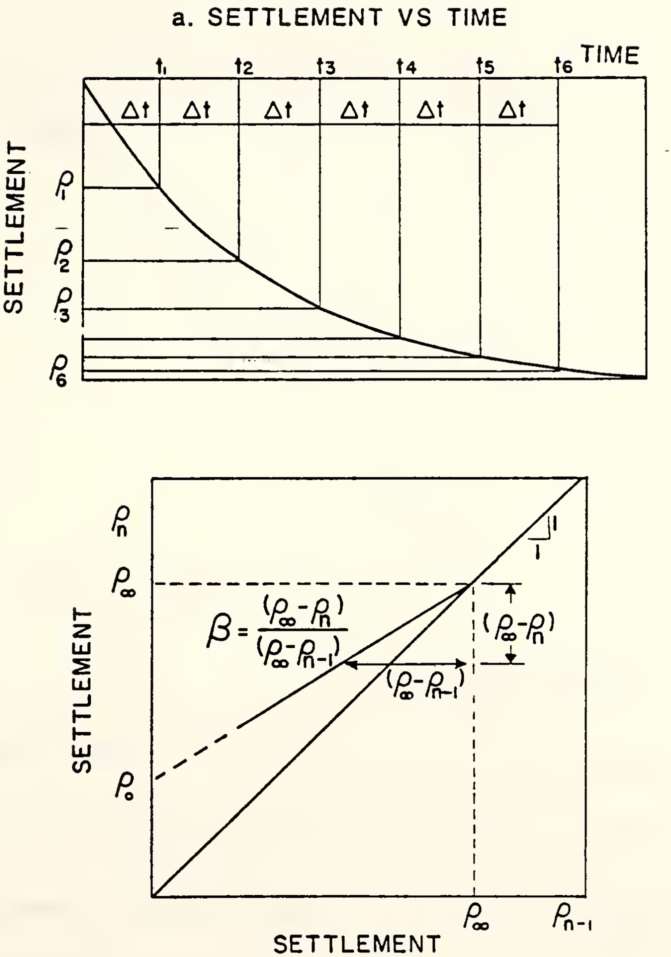
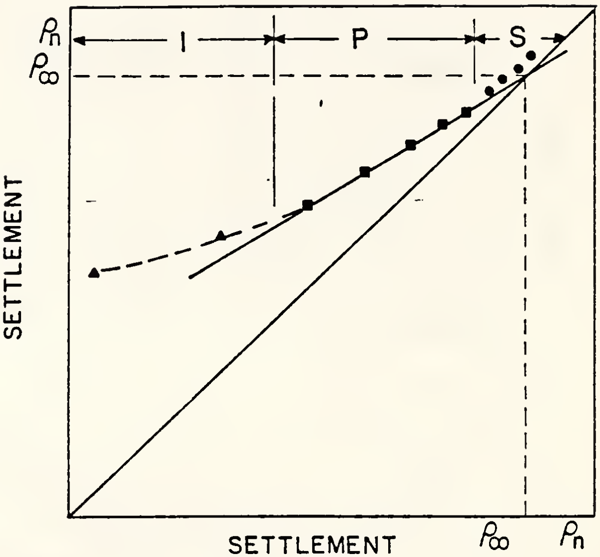


Figure 7.8 Example of Asaoka's (1978) Construction.



ZONE	SYMBOL	STATE
I	▲	INITIAL STAGE OF CONSOLIDATION (C OR C DECREASING)
P	■	PRIMARY CONSOLIDATION WITH CONSTANT C OR C
S	●	SECONDARY COMPRESSION

Figure 7.9 Possible Deviations from Asaoka's Construction. From Orleach (1983).

From which:

$$\rho_j = \beta_o + \beta_1 \rho_{j-1} + \beta_2 \rho_{j-2} \tag{7.6a}$$

and

$$\rho_{j-1} = \beta_o + \beta_1 \rho_{j-2} + \beta_2 \rho_{j-3} \tag{7.6b}$$

Subtracting 7.6b from 7.6a:

$$\rho_j - \rho_{j-1} = \beta_1(\rho_{j-1} - \rho_{j-2}) + \beta_2(\rho_{j-2} - \rho_{j-3}) \tag{7.7}$$

Letting $\Delta\rho_j = \rho_j - \rho_{j-1}$,

$$\Delta\rho_j = \beta_1 \Delta\rho_{j-1} + \beta_2 \Delta\rho_{j-2}$$

or

$$\frac{\Delta\rho_j}{\Delta\rho_{j-2}} = \beta_1 \frac{\Delta\rho_{j-2}}{\Delta\rho_{j-2}} + \beta_2$$

Hence, by calculating:

$$x_j = \frac{\Delta\rho_{j-1}}{\Delta\rho_{j-2}} \text{ and } y_j = \frac{\Delta\rho_j}{\Delta\rho_{j-2}}$$

the coefficients β_1 and β_2 can be obtained by fitting a straight line $y = \beta_1 x + \beta_2$ to the points (x_j, y_j), $j = 2,3,4$, etc. using the least squares method. Once β_1 and β_2 are known, β_o is the average of $(\rho_j - \beta_1\rho_{j-1} - \beta_2\rho_{j-2})_j$=2,3,4, etc. For n>2, the same procedure applies. In this case,

$$\frac{\Delta\rho_j}{\Delta\rho_{j-3}} = \beta_1 \frac{\Delta\rho_{j-1}}{\Delta\rho_{j-3}} + \beta_2 \frac{\Delta\rho_{j-2}}{\Delta\rho_{j-3}} + \beta_3 \tag{7.8}$$

By defining:

$$(\nu_1)_j = \frac{\Delta\rho_j}{\Delta\rho_{j-3}}, \quad (\nu_2)_j = \frac{\Delta\rho_{j-1}}{\Delta\rho_{j-3}} \text{ and}$$

$$(\nu_3)_j = \frac{\Delta\rho_{j-2}}{\Delta\rho_{j-3}}, \text{ we get}$$

$$(\nu_3)_j = \beta_1 (\nu_2)_j + \beta_2 (\nu_1)_j + \beta_3 \tag{7.9}$$

Similarly,

$$(\nu_3)_{j-1} = \beta_1 (\nu_2)_{j-1} + \beta_2 (\nu_1)_{j-1} + \beta_3 \tag{7.10}$$

From which

$$(\nu_3)_j-(\nu_3)_{j-1} = \beta \, [(\nu_2)_j-(\nu_2)_{j-1}] + \beta_2 \, [(\nu_1)_j-(\nu_1)_{j-1}]$$

or

$$\frac{(\nu_3)_j-(\nu_3)_{j-1}}{(\nu_1)_j-(\nu_1)_{j-1}} + \beta_1 \left[\frac{(\nu_2)_j-(\nu_2)j-1}{(\nu_1)_j-(\nu_1)_{j-1}}\right] + \beta_2$$

From which β_1 and β_2 can be found. β_3 can be found from Eq. 7.9 and β_1 from Eq. 7.8.

After identifying the parameters, the final settlements are calculated as

$$P_j = \frac{\beta_o}{1 - \sum_{s=1}^{n} \beta_s}$$

for the n=2 and the n=3 model. When these are more or less equal, sufficient data are available for calculating settlements at any time using the equation:

$$\rho_j = \rho_j + \sum_{s=1}^{3} G_s (R_s)^j \qquad (7.11)$$

where R_s, s=1,2,3 are the roots of

$$R^3 - \beta_1 R^2 - \beta_2 R - \beta_3 = 0$$

and the constants G_s, s=1,2,3 are calculated from the previously obtained settlement values by the method of least squares. Using Eq. (7.11), the settlement to be expected for the lifetime of the structure can be calculated, and a surcharge of about two to three times the design load is applied until this settlement is exceeded.

As can be seen, the method for n=2 and 3 is not as simple as the graphical method. Asaoka and Matsuo (1980) state the model gives good results for complex boundary conditions. However, since it is unlikely that the site can remain unsurcharged, with just the design load acting on it for purposes of taking data to fit the model, it is of not much value at present for soils such as peats and mucks, where surcharging is essential.

Matsuo and Asaoka (1980) suggested a method by which data from only the construction phase are sufficient to make future predictions. In this case, future settlements under design loads can be predicted using data obtained during construction. Hence, no halt in loading will be required on reaching the design load, in order to predict future movements, and the surcharge load can be directly applied. The method, however, is not simple and requires that readings be taken during the construction process which itself may not be very practical. In general, however, the Gibson-Lo method is, in terms of application, more suitable than the method of Asaoka and Matsuo, for the case where a surcharge is to be used. Greater accuracy of prediction can be realized by ensuring that no lateral displacement or plastic strain occurs, rather than simply increasing the sophistication of the analysis.

The Gibson-Lo model can be applied with a fair amount of accuracy to other stress levels. Asaoka's method, however, is valid only at the present load level and predicting settlements at any other stress level is not possible. Thus, while it is capable of predicting settlements under the surcharge load (based on observations taken when the material is subject to the surcharge load), no information is available to predict settlements under the design load. Because of this, it is not possible to calculate the amount and duration of surcharging. In other words, for materials like peat and muck, where surcharging is required, the Gibson-Lo method still remains the preferred method.

THE DESIGN METHODOLOGY

Based on the previous discussion, it is now possible to formulate a design methodology for embankments on amorphous peats and mucks. Once it has been decided that an embankment is to be constructed on amorphous peat or muck, a classification of the peat or muck must be accomplished. Once this is done, a rough idea of the properties involved can be obtained. Next, using a probe, such as the Mackintosh probe [Clayton et al (1982)] an idea of the depth of the materials can be obtained. Once this is available, details of the subsoil are obtained in order to determine if any soft clays are present, which can cause problems of the sort mentioned earlier.

Next, disturbed samples are taken from areas likely to be subject to stress (areas that might be occupied by the embankment, berms, etc.). From the water contents and the ash contents of the peats and mucks, the variability of the deposit is estimated and the areas with the highest water and lowest ash contents identified. Next, an effort is made to route the proposed embankment through the better areas if possible.

Now, samples that are undisturbed as far as possible are taken - block samples from the surface, and using say a split barrel sampler with a sectioned inner lining at depth. These samples should be tested as early as possible. using the methods mentioned in Chapter V. The tests are to determine the value of C_c, C_α, C_α/C_c, k, the undrained shear strength, and the rheological parameters a, b, and λ of the Gibson-Lo model.

Tests should be run on as many samples as possible, taken from over the area subject to stress. On the basis of the disturbed tests, the deposit can be divided both in plan and in depth into localized regions of roughly similar organic contents, and samples taken from each. Once the test data are available, an estimate of the amount of settlement likely to occur is made using the simplified method of Chapter IV and Eqs. 4.39 to 4.42.

Initial settlements can be computed using the analysis normally used for soft clays. Once the settlement is roughly calculated, accounting for the additional stress required to bring the embankment to grade, the geometry is to be investigated to decide whether berms will be required. Due to the high variability of the material, the strengths both drained and undrained vary, and so any stability analyses should account for this. One way of doing this is to use a program such as STABL, making use of a probabilistic method similar to that proposed by McGuffey et al (1982) and described in detail by Goodman, Chameau and Lovell (1983).

When using this type of program, a spreading type failure should also be analyzed. If the underlying material is soft clay with a low permeability, drainage of the clay should be ensured. It must be remembered that for embankments on peats, the underlying soft clay is usually the cause of any failure. For mucks, it may be that the permeability is very low if the clay content is high. Hence, the muck may, depending on the k values obtained, require drainage.

From the rheological parameters, and the maximum settlement expected during the lifetime of the structure, the value of the stress increment term $\Delta\sigma$ in the Gibson-Lo model (Eq. 7.1) can be calculated for a convenient value of time of application t, from which the height of surcharge can be determined. To account for factors such as strain rate dependence of the rheologic parameters, departure from one-dimensional conditions and various approximations or assumptions, this surcharge can be increased to say 1.3 times the calculated value.

Once the preliminary calculations, as regards the height and geometry of both the actual embankment and the surcharge load are made, field instrumentation to determine vertical and lateral movements is carried out. When carrying out instrumentation it must be remembered that strains will be large, and that at least initially, the soil may be soft enough to flow around the instruments. After placing the instrumentation, embankment construction is begun, taking care to monitor the lateral deformations of the peat or muck, and the underlying soft clay if any. Based on these observations, the rate of constructing is controlled in accordance with the procedures described earlier.

When the surcharge height is reached, vertical settlement observations are taken regularly, the field values of the rheological parameters are obtained, and these are used to recalculate settlements under the design load and for the design life. These calculations are repeated as more and more settlement observations become available, until the values of a, b, and λ reach more or less constant values. After this settlement has occurred, the surcharge is removed, and the embankment brought to grade. Bringing the embankment back to grade could require that some of the surcharge remain in place.

CONCLUSION

— Apart from increasing the sophistication of the analyses, significant improvements in the accuracy of the existing methods can be achieved by ensuring that lateral displacement and strain softening do not occur. Purely predictive models may not be justifiable, except for test fills where it is possible to do a large number of accurate tests. Observational methods are far better in this respect, and a combination of the two approaches is suitable for materials like peat and muck. Though formulated in 1961, the Gibson-Lo model still remains one of the better observational methods available to determine duration and/or amount of surcharge. It is, therefore, the preferred method for settlement prediction. The method proposed by Sekiguchi and Shibata (1979) can be used to predict the rate of loading, the ultimate bearing capacity, and hence, the immediate factor of safety. When used in conjunction with each other the Sekiguchi and Shibata (1979) method for construction and the Gibson-Lo method for settlement prediction form a complete observational methodology for peats and mucks.

CHAPTER VIII - SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

SUMMARY

This chapter summarizes all that has been discussed thus far, draws conclusions, and makes recommendations as to future work to be done on these materials.

Chapter II revealed that considerable misuse of the word ´peat´ occurs in the literature, as a result of which the same material (peat) seems to exhibit widely differing properties. This confusion exists because there is no consensus on the type of classification system to be standardized. This problem is further compounded by the fact that the material is truly variable. Chapter II suggested that for peats, rather than test a large number of samples, a better approach might be to accurately classify the material and use correlations between peat type, and the properties. A technique of classification using fuzzy sets was suggested, along with a comprehensive classification system for peats. Also described in this chapter was the classification system presently used for the mucks.

Chapter III studied the best method of subsurface exploration of these materials. In view of the difficulty in sampling and testing undisturbed samples, and given the inherent soil variability, it was felt that rather than rely on a few costly, accurate tests, a better method might be to limit these tests to the areas of primary interest and to do a larger number of lesser quality tests on the other areas of the deposit.

Chapter IV examined the question of nonlinear finite strain consolidation. Two theories were examined in detail. One was the Berry-Poskitt model which was developed in this case specifically for amorphous peats, while the other was the general model of Mesri and Rohksar. It was found that these models, particularly the latter, were capable of predicting laboratory behavior. However, the laboratory situation is quite different from the field situation, the more so for materials like peat and muck. This is true both for the material properties and also the boundary conditions. Because of this, these purely predictive models are suitable only for projects like test fills, where detailed exploration is appropriate. Hence, Chapter IV suggested that as far as prediction was concerned, a simplified model would suffice to obtain an estimate of settlement, after which the observational model can be used.

Chapter V gave details of a comprehensive testing program, carried out at Purdue University and involving soils with a wide range of organic contents. Three soils were sampled from Indiana, and subjected to consolidation, creep, permeability tests and shear strength tests. From the consolidation tests, it was seen that the e-log $\bar{\sigma}$ curve was not linear. Creep tests conducted on undisturbed samples revealed that like many clays the e-log t relation was linear. The Gibson-Lo model was studied and it was concluded that where the concerned parameters were obtained at or near 1.5 to 2.5 times the design stress level, and, if the design stress level was low, the model worked well. The permeability tests showed that, like some inorganic soils, the e vs. log k relation could be approximated by a straight line.

Finally, the shear strength tests on samples consolidated under K_o conditions revealed that though the index tests indicated a high variability, as far as the shear strength was concerned, this was not the case. A unique failure envelope was easily definable in p'-q space for both the normally consolidated and the overconsolidated samples. Also the failure envelope for the normally consolidated soil passed through the origin and was concave upwards at values of p' greater than about 140 kPa. This showed that the strength of these materials is purely frictional, and it also explains the high values of the friction angle observed by other researchers. It was also noticed that the stress paths followed by the normally and the overconsolidated samples agreed quite well with the critical state model.

Indirectly, the great difference in c_v for the natural material and for the material obtained from slurry, implied that the natural material had a definite fabric which influenced most properties. Finally, the creep tests were found to be insensitive to minor fluctuations in temperature.

Chapter VI described the various methods of construction on organic soils - displacement, floatation and surcharge. The use of geotextiles was investigated, and it was found that they were of use in minimizing local failure and reducing the height of fill required. Next, embankment behavior was considered in the light of the effective stress analysis/yield envelope approach. It was concluded that differences between predicted and observed behavior, particularly during the early stages, was more a result of displacement of the peat or muck and strain softening, rather than the method of analysis lacking sophistication. It was shown that preloading reduced settlements under the design load, and also reduced the creep rate.

Next, the behavior of embankments on peats and mucks was examined, along with their behavior on clays, and it was concluded that considerable drainage would be occurring during construction. There were very few settlement records, and no lateral displacement records. Based on the observed behavior, an effective stress path/yield envelope model for peats was proposed. The various failure modes were studied, and it was found that underlying soft clay was usually the cause of the failure. Also examined in this chapter was the use of berms, drains and lightweight fills, and it was found that each was useful depending on the circumstance. Also examined were the problems involved in widening a road on peat.

Chapter VII proposed a design methodology that used the simplified predictive model described in Chapter IV to obtain an initial estimate of settlements, and probabilistic slope stability to determine the geometry. Once these estimates are available, the observational procedure is used, with the method of Sekiguchi and Shibata (1979) recommended for the immediate case and the Gibson-Lo model recommended for the long term case.

CONCLUSIONS

On the basis of the investigation, the following conclusions were achieved.

1) Considerable misuse of the word 'peat' occurs in the literature. This is due to the lack of consensus on the type of classification system. The von Post classification system is the best available.

2) Rather than test a large number of samples, a better approach may be to classify the material accurately, and then use correlations between the peat type and the properties.

3) Fuzzy sets can be helpful in the visual/manual classification of peats.

4) Site investigation, if used properly can minimize the amount of testing required.

5) Sophisticated consolidation theories are available to predict laboratory behavior. However, these theories are suitable only for test fills.

6) The e - log$\bar{\sigma}$ curve was shown to be nonlinear.

7) The e - log t relation for creep tests was linear.

8) The Gibson-Lo model was shown to work well, when the concerned parameters were obtained at or near 1.5 to 2.5 times the design stress level, and if the design stresses are low.

9) The index tests give results with a large scatter; however, the p$'$-q curve is unique. The strength of these materials is purely frictional. The failure envelope is concave upwards at higher stress levels.

10) The effective stress paths for these materials in undrained shear shows good correspondence with that of soft clays.

11) Fabric plays a major role for such materials, as indicated by the reduced c_v values of the samples prepared from slurry.

12) Creep tests were found to be relatively insensitive to minor fluctuations in temperature.

13) Differences between predicted and observed behavior, particularly during the early stages of construction, were more a result of displacement of peat and muck, and strain softening, rather than the method of analysis lacking sophistication.

14) Considerable drainage occurs during construction on peats and mucks.

15) The effective stress path/yield envelope concept can be used to qualitatively explain the behavior of embankments on these materials.

16) Very few vertical settlement and lateral displacement records exist for embankments on these materials.

17) Failures on these materials occur usually when the underlying material is soft clay.

18) Berms, drains and lightweight fill were found to be useful, depending on the circumstance.

19) The observational methods were found to be very useful for construction on these materials.

20) Lastly, a complete design methodology was provided, giving details about the geometry, loading rate, settlements to be expected. This is outlined in Chapter VII, pages 214 to 218.

RECOMMENDATIONS

There exist deficiencies in our knowledge as regards certain phenomenon exhibited by peats and mucks. This section attempts to identify these areas.

1) Once a peat or muck is accurately classified, and if a suitable data base exists, the need for detailed experiments is reduced. Improving the classification system and creation of this data base will be of help to an engineer who has to deal with this material. Work has been initiated at Purdue University to see how the existing systems work. A questionnaire has been prepared for this, and is included in Appendix C. Further work in this area, especially regarding the use of fuzzy sets for classification, is recommended.

2) Testing undisturbed samples is not simple, but even before that, there are some questions about the behavior of samples consolidated from a slurry that are not easy to answer. For ex. Why does the K_o value continue to decrease after consolidation is over? Why are the K_o values generally so low, especially for the overconsolidated soils? Why is the stress path for samples known to be normally consolidated similar to that of a lightly overconsolidated soil? Why is the failure envelope concave upwards and also why is it unique when the index properties are highly variable? These and other similar questions remain to be answered and further work is recommended.

3) Asaoka's method offers significant advantages for single stage construction. It is recommended that research be done to make it useful for surcharging also.

4) The final recommendation would be the construction of a test embankment on an organic material, with suitable instrumentation to monitor vertical movements, lateral movements and pore pressures. The validity of the concepts suggested in Chapters VI and VII can then be studied.

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